VOL. XXXVIII. No. 7 JULY, 1960

THE STRUCTURAL ENGINEER

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Some Factors in the Shear Strength of Reinforced Concrete Beams
by Dr. A. M. Neville (Associate-Member) and E. Lord

Deflections of Plane Frames at the Point of Collapse
by Professor B. G. Neal

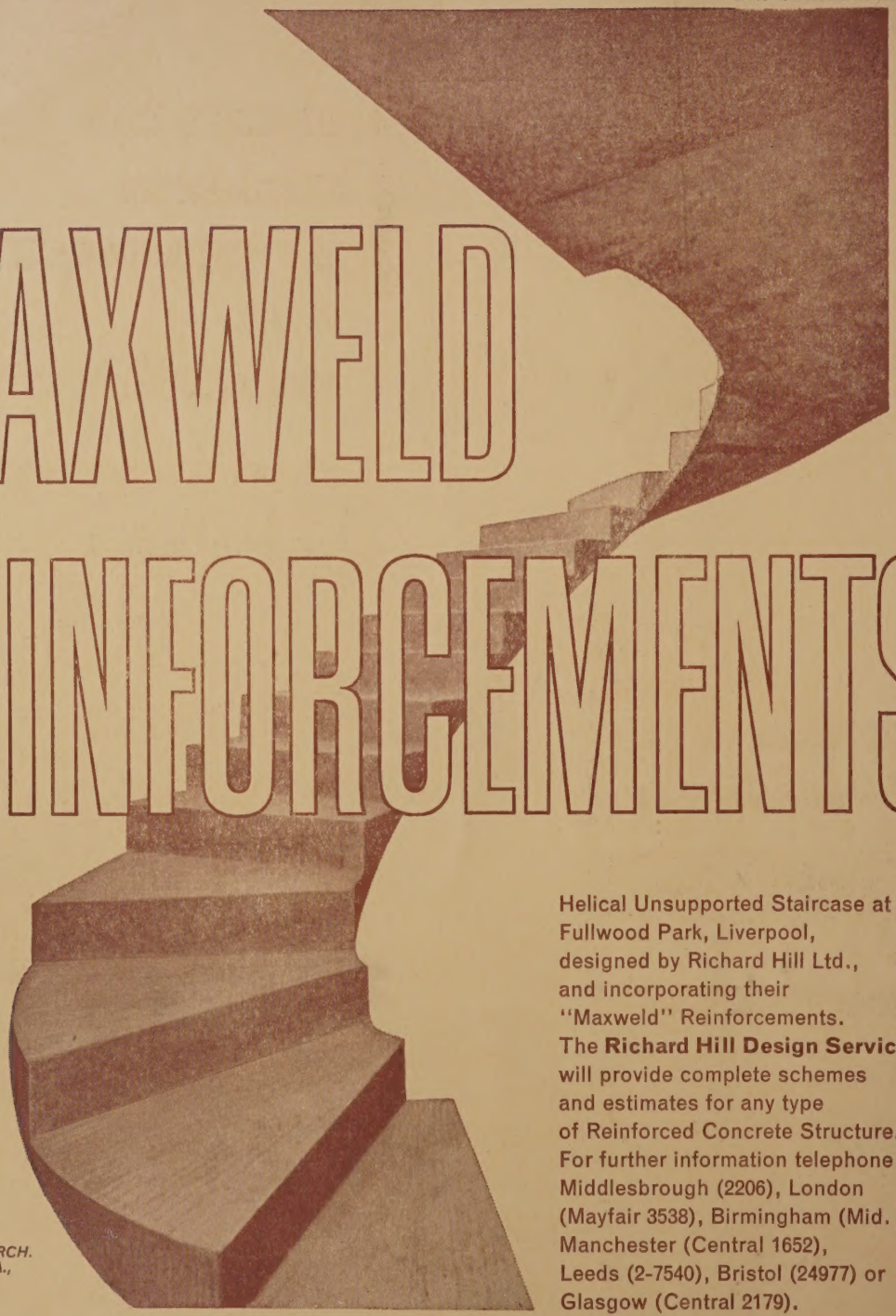
Practical Design of Pile Groups
by A. Kroie (Associate-Member)

The Structural Engineer in the Field of Atomic Energy
Discussion on the Paper by T. C. Waters (Delegate Member of Council).

A Convergence Technique for Determining the Elastic Critical Load of Rigidly Jointed Trusses
Written Discussion on the Paper by Dr. Arthur Bolton

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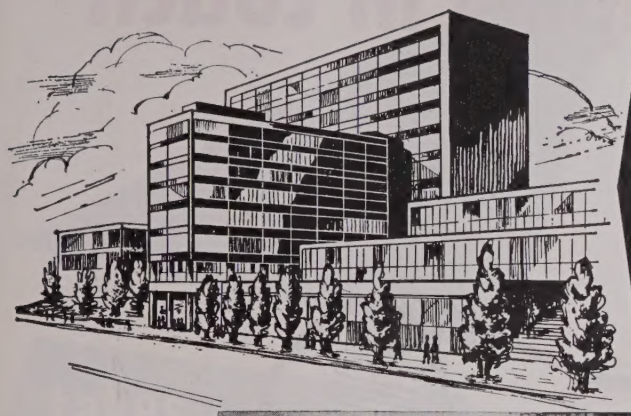
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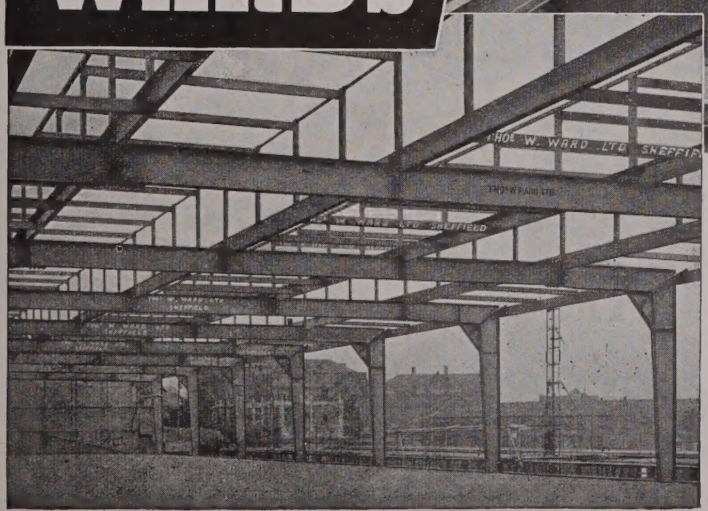
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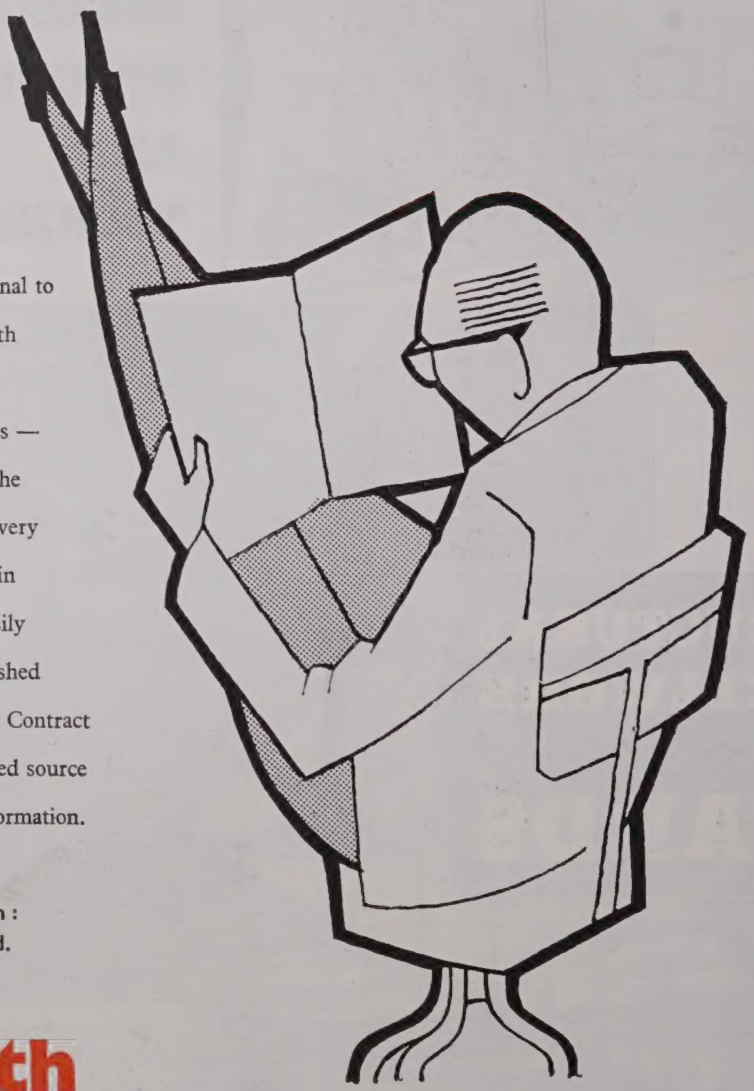
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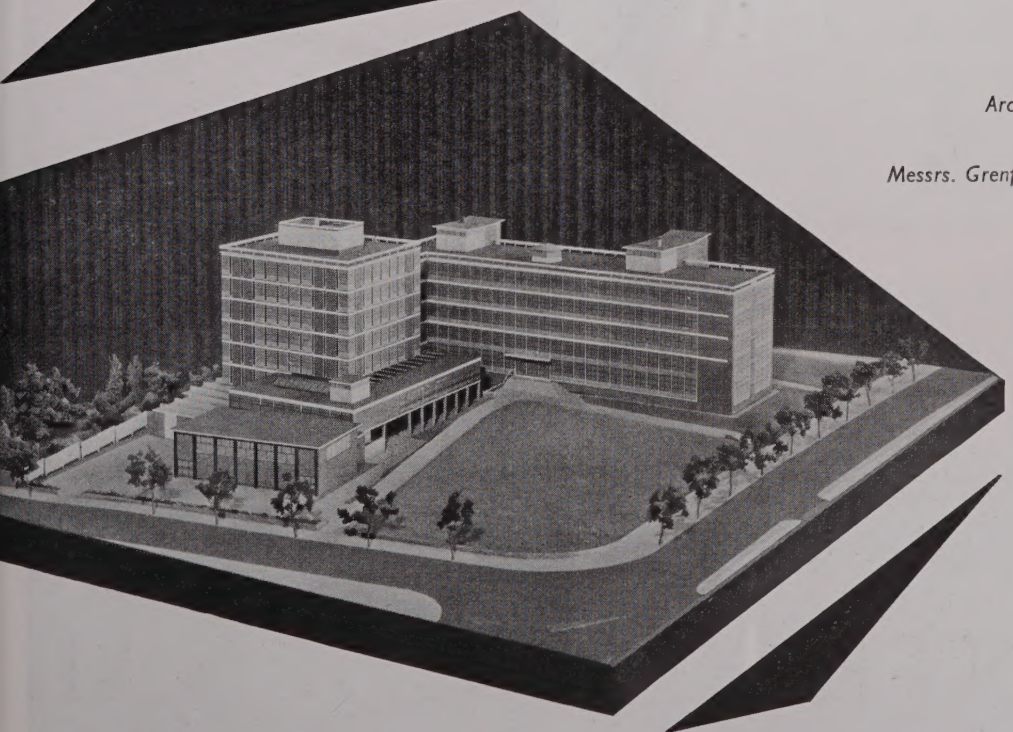
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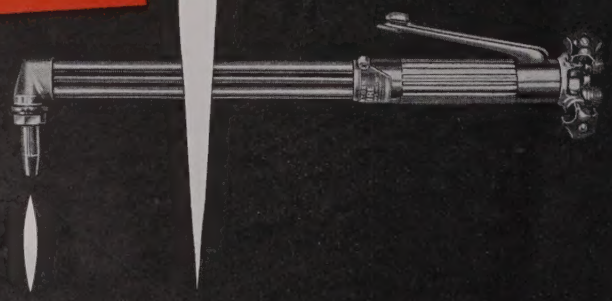
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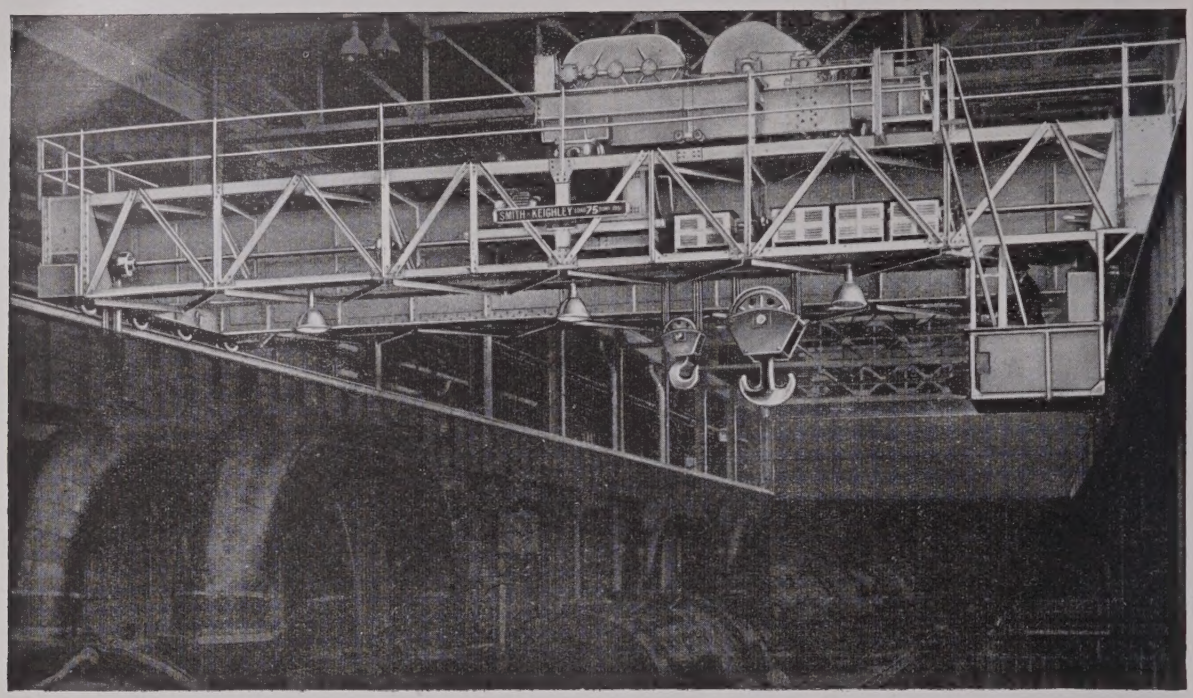
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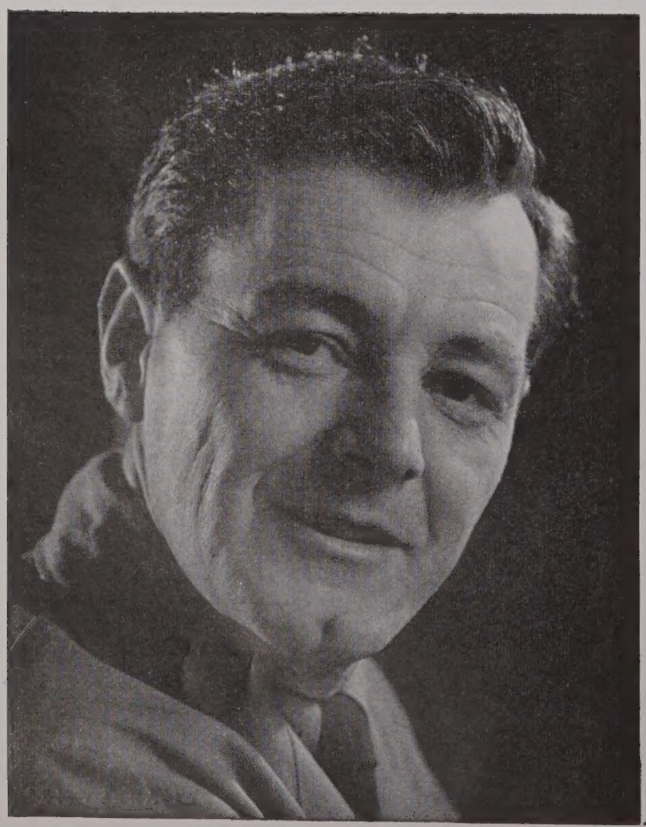
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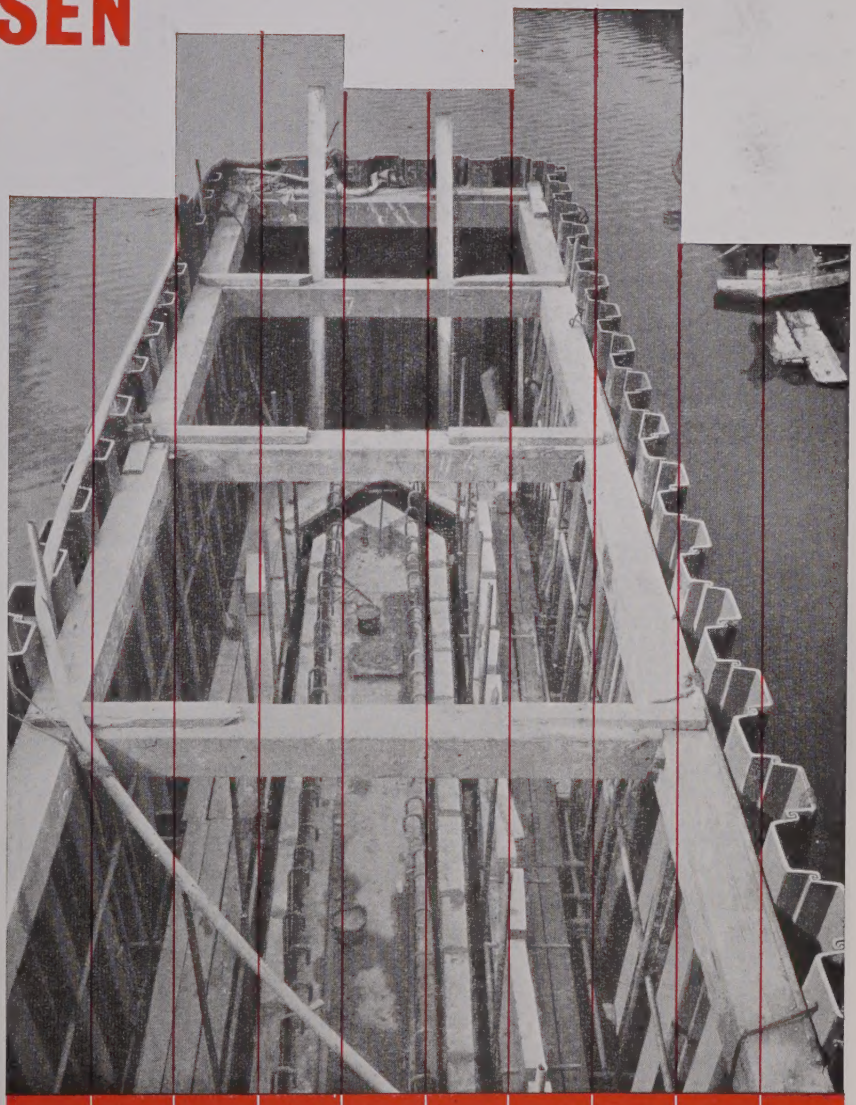
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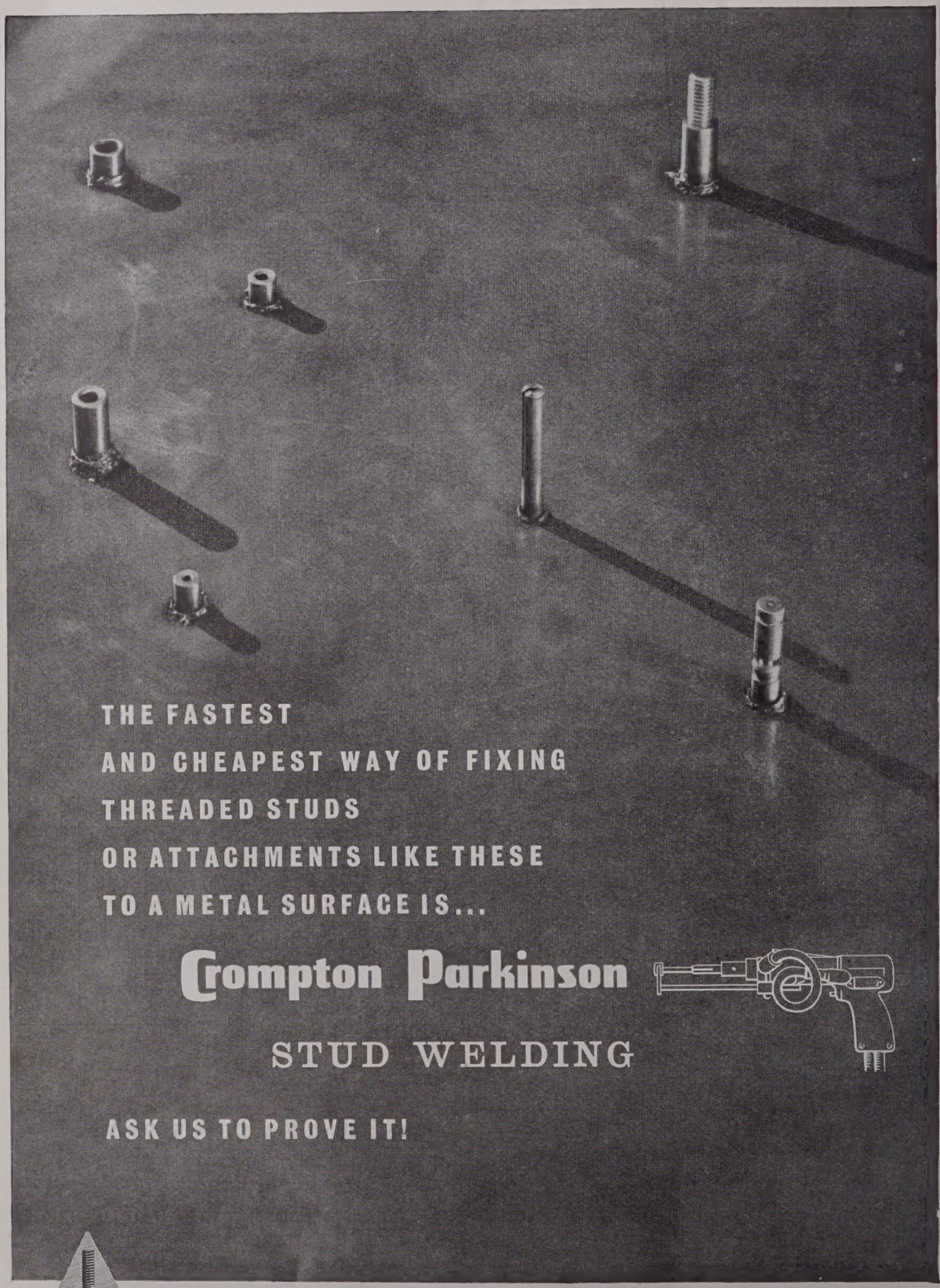
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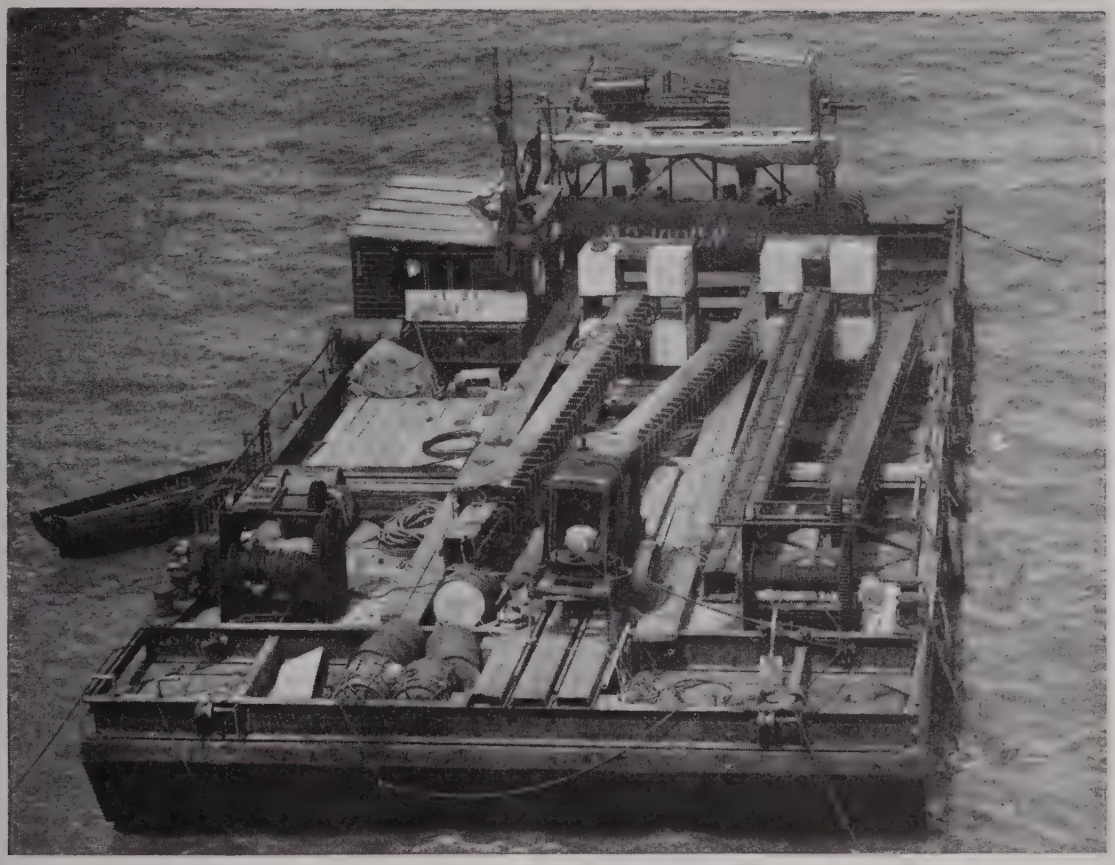
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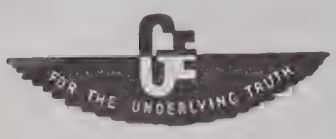
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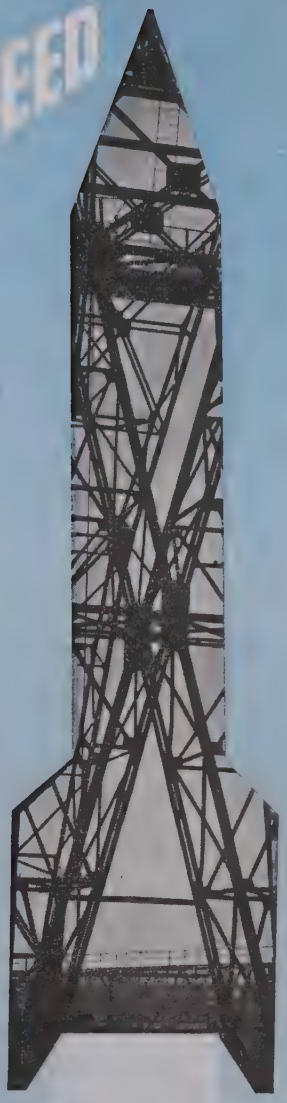
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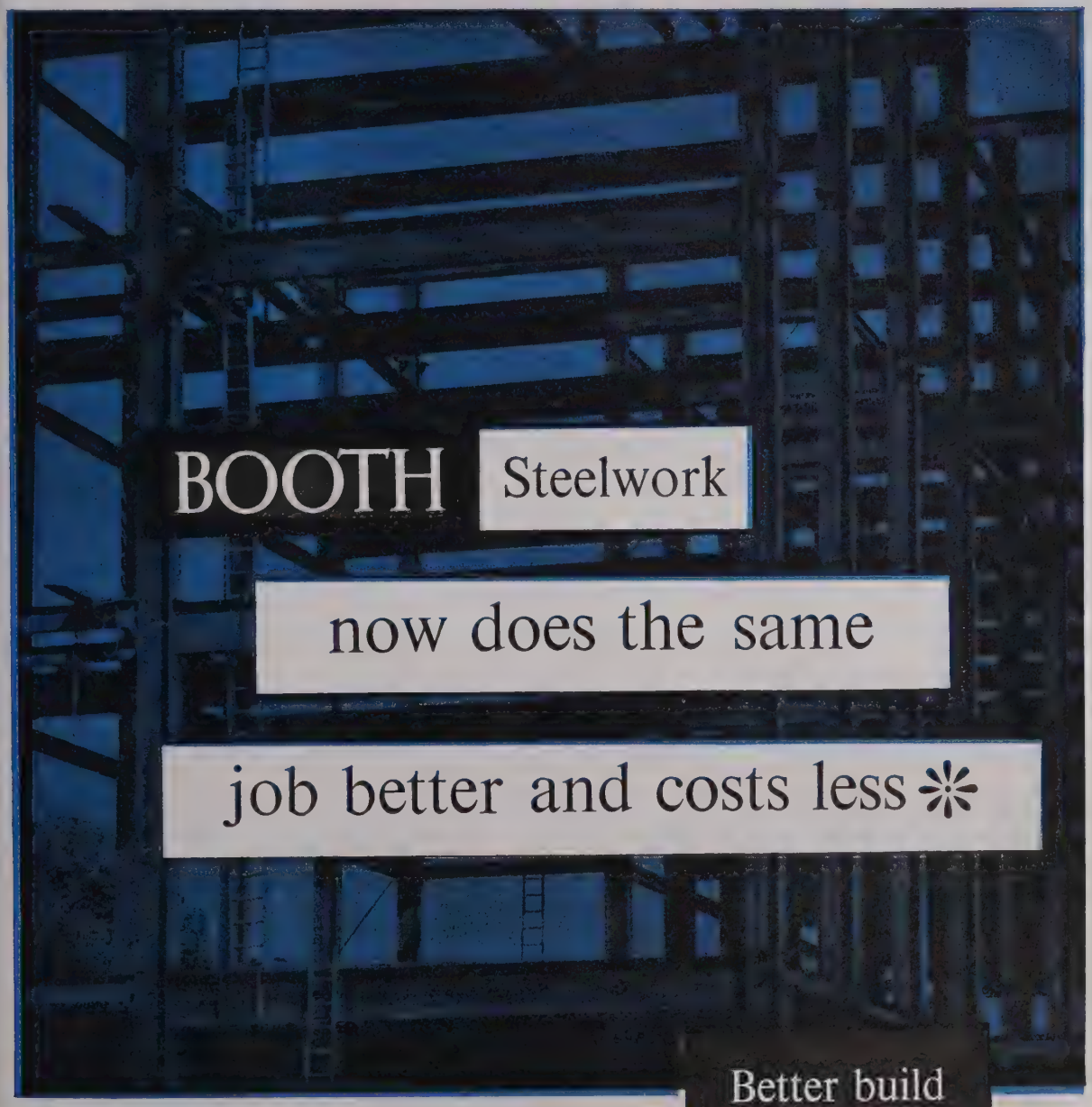
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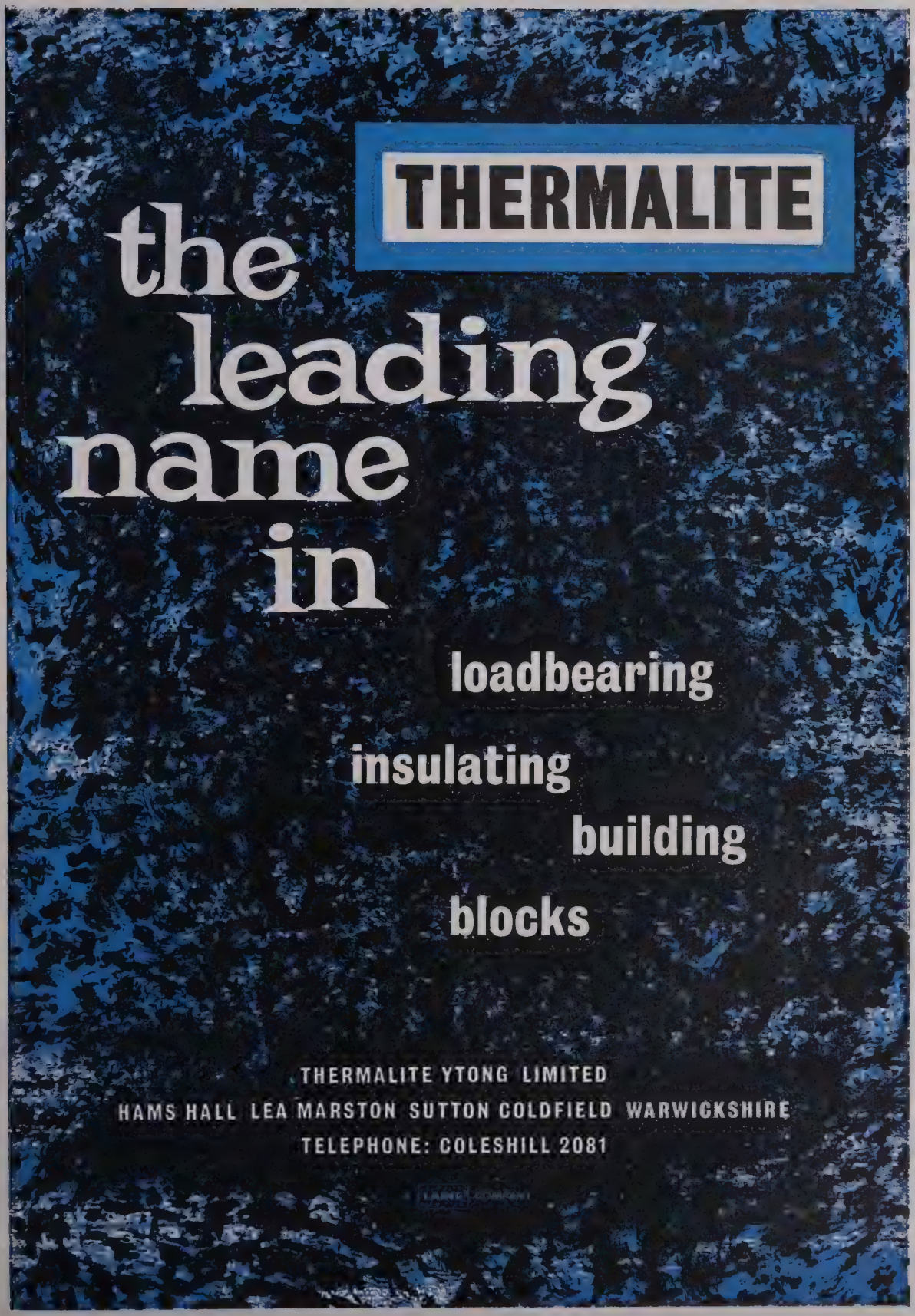


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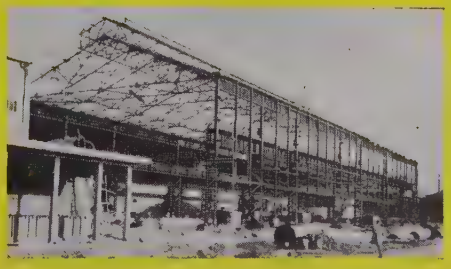
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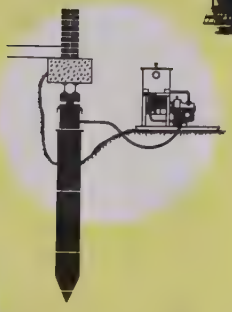
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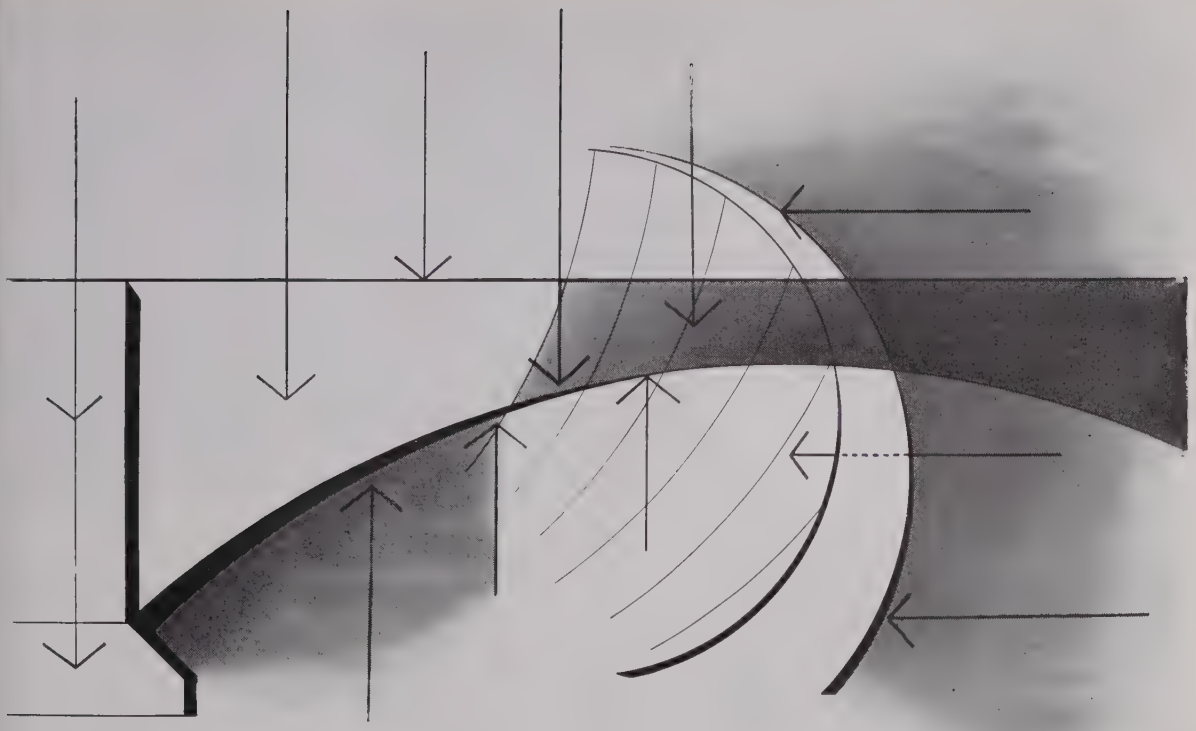


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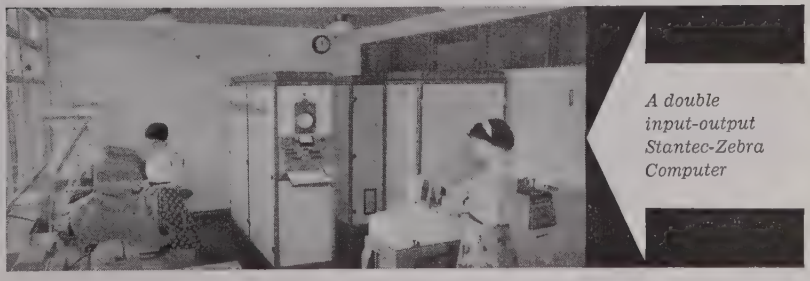
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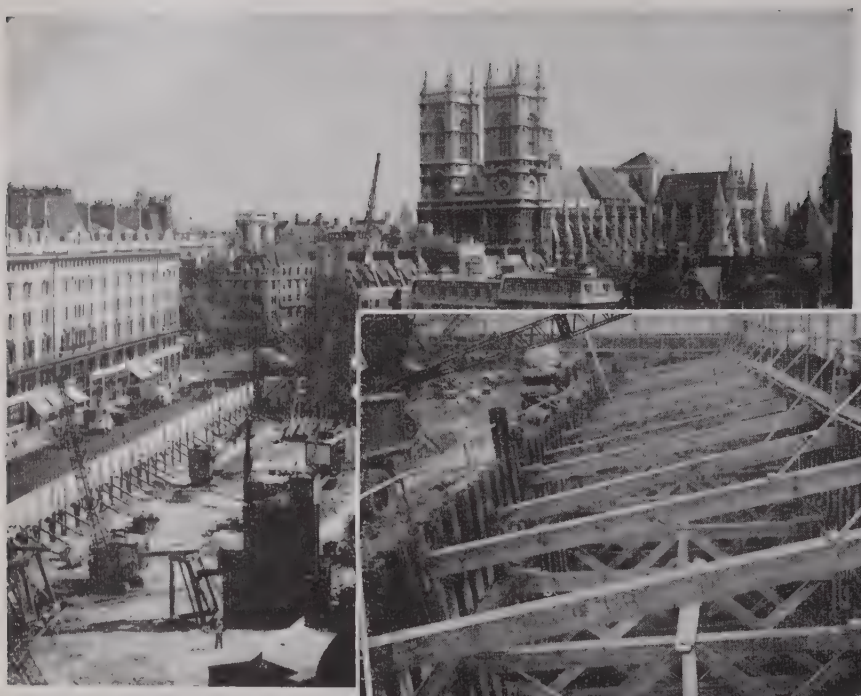


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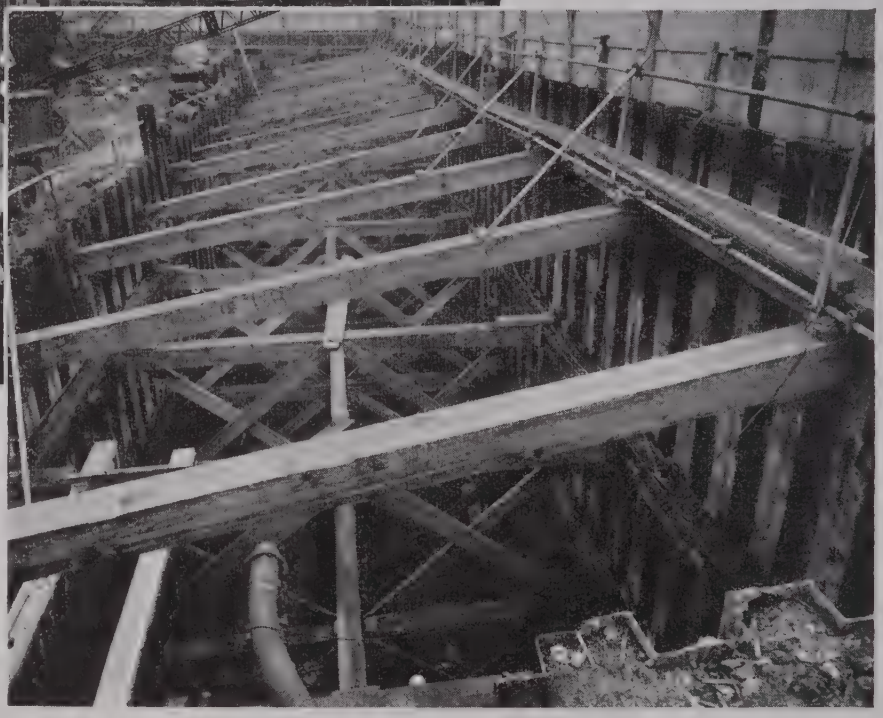
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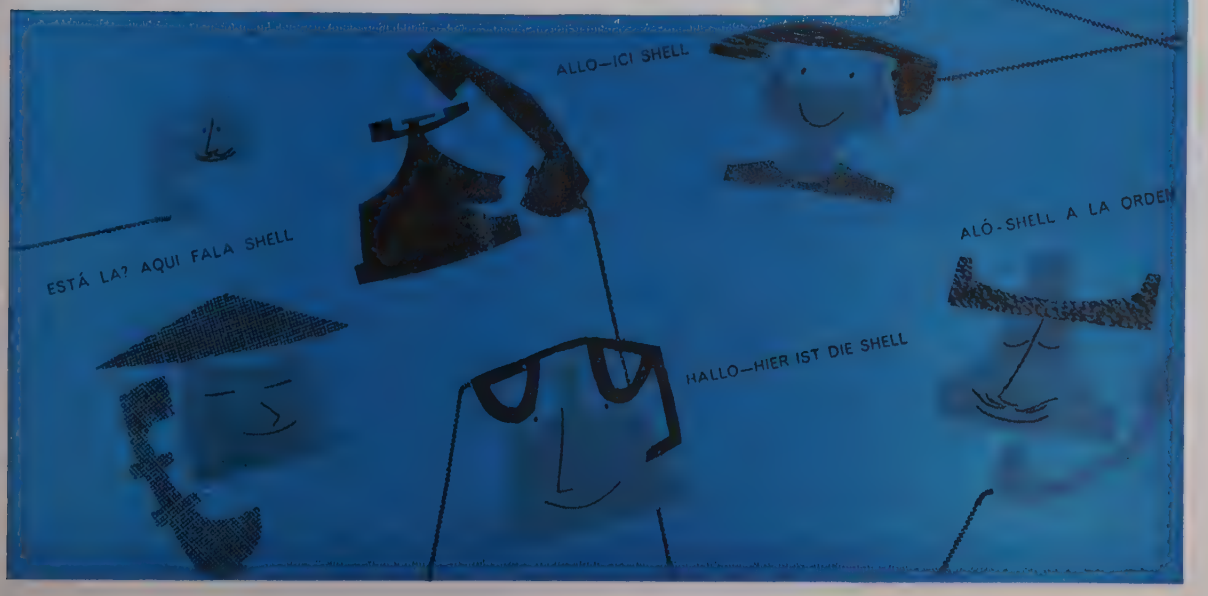
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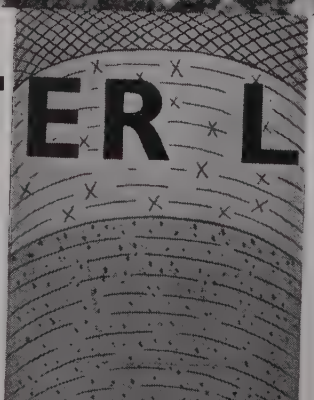






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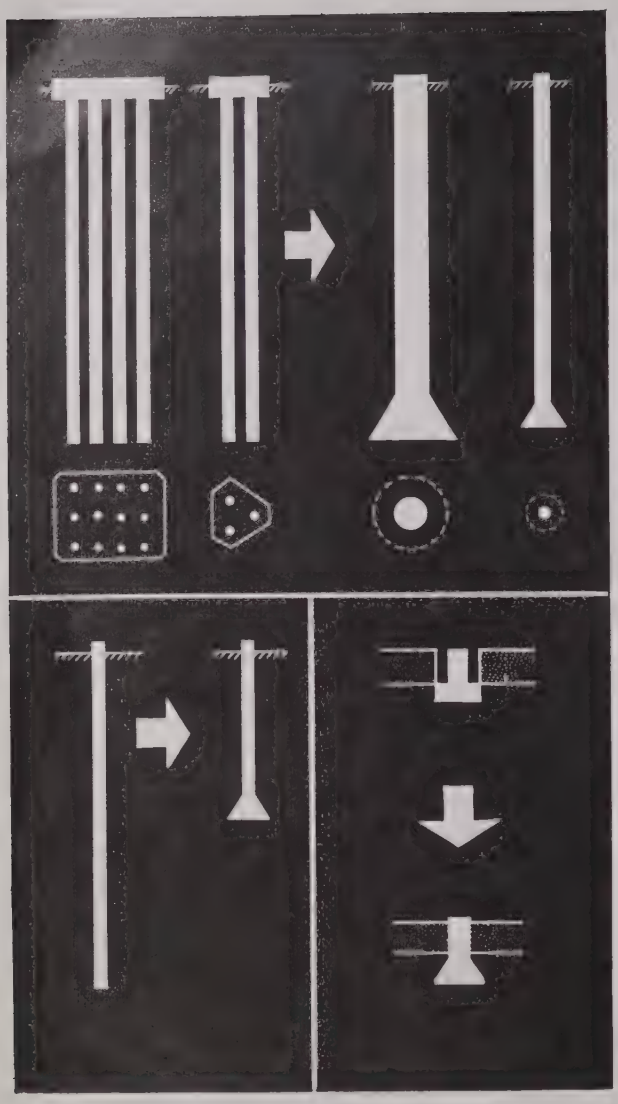
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CONTENTS

PAGE

SOME FACTORS IN THE SHEAR STRENGTH OF REINFORCED CONCRETE BEAMS 213
By A. M. Neville, M.C., M.Sc., Ph.D., A.M.I.Struct.E., A.M.I.C.E., and E. Lord, M.Sc.
The Structural Engineer, July, 1960

DEFLECTIONS OF PLANE FRAMES AT THE POINT OF COLLAPSE 224
By Professor B. G. Neal, M.A., Ph.D., M.I.C.E.
The Structural Engineer, July, 1960

PRACTICAL DESIGN OF PILE GROUPS 230
By A. Kroie, A.M.I.Struct.E.
The Structural Engineer, July, 1960

THE STRUCTURAL ENGINEER IN THE FIELD OF ATOMIC ENERGY 235
Discussion on the Paper by T. C. Waters, M.I.Struct.E. (Delegate Member of Council)
The Structural Engineer, July, 1960

A CONVERGENCE TECHNIQUE FOR DETERMINING THE ELASTIC CRITICAL LOAD OF RIGIDLY JOINTED TRUSSES 239
Written Discussion on the Paper by A. Bolton, M.Sc.(Tech.), Ph.D.
The Structural Engineer, July, 1960

BOOK REVIEWS 223, 234, 238

INSTITUTION NOTICES AND PROCEEDINGS 242

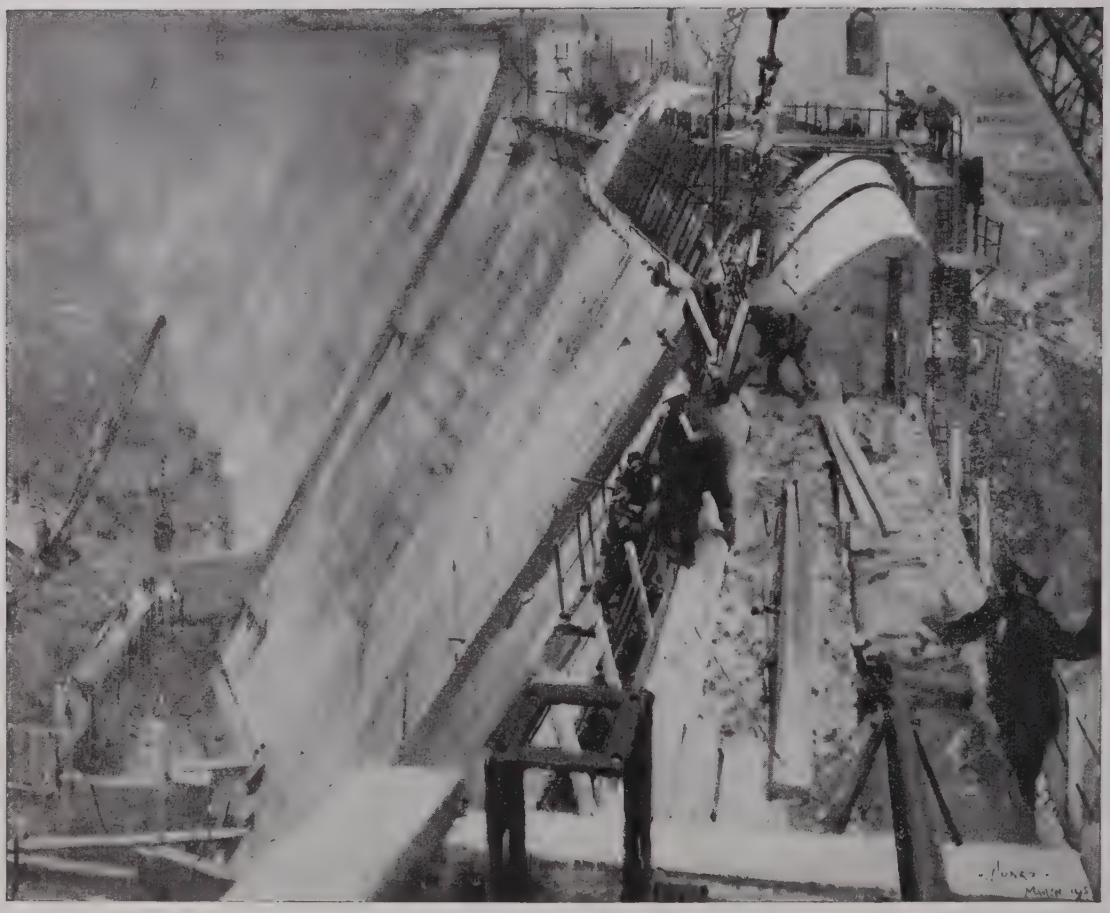
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Some Factors in the Shear Strength of Reinforced Concrete Beams

by

A. M. Neville, M.C., M.Sc., Ph.D., A.M.I.Struct.E., A.M.I.C.E. and E. Lord, M.Sc.

Summary

The different types of shear failure of reinforced concrete rectangular and T-beams are described in order to show that the shear strength depends on factors additional to those recognized by the design formulae. Results of experiments on the influence on shear strength of the size of the compression zone of the beam and of hooks on plain and deformed bars are presented. Tests on rectangular beams with varying amounts of compression reinforcement suggest that this does not affect the shear capacity of the beam.

Introduction

During the last few years it has been increasingly recognized that the strength in shear of a reinforced concrete beam without web reinforcement is not a simple function of the width of the beam b and its lever arm l_a, as would appear from the standard code formula. Likewise, there is a considerable doubt about a correct method of calculating the web reinforcement required, and in fact, different codes make different assumptions regarding the proportions of shear resisted by the concrete and by the web reinforcement.

The problem of shear is complicated by the basic difference between the distribution of forces in an uncracked beam and in a beam at failure.

The classical formulae apply to working loads and under the action of these a properly designed beam should be free from diagonal tension cracks. Thus the criterion should be that the principal tensile stress anywhere in the beam is below the tensile strength of concrete, or in design below a permissible value. This applies to beams without and with web reinforcement alike, for no appreciable force is registered in the stirrups until after the diagonal tension crack has formed.[1]

While the requirements for a crack-free beam under the working load should be satisfied in the design, it is also essential to investigate the behaviour of the beam at failure in shear. The ultimate strength approach to flexure design is being increasingly adopted, and both the current (1957) British Code of Practice and the American Concrete Institute Standard Code permit the use of the load factor method. It would be illogical for one and the same beam to be designed by the plastic method as far as flexural stresses are concerned and yet to be viewed from the elastic standpoint only with respect to shear. Moreover, such a procedure may result in a grossly inadequate factor of safety, as shown by Taub and Neville[2].

The cause of this situation is the avowed or tacit assumption of the truss analogy in the current design formulae of all the codes. That the truss analogy is not valid has been demonstrated conclusively by Neville and Taub[3]; in particular, it was shown that the use of small stirrups at a small pitch results in a considerably higher ultimate load than when the beam is reinforced with larger stirrups at a larger pitch, both systems of web reinforcement being equivalent in terms of the ratio $\frac{A_w}{s}$, arising from the Code formula :

$$\frac{A_w}{s} = \frac{Q}{P_{st} l_a} \quad . \quad . \quad . \quad . \quad . \quad . \quad (1)$$

where, A_w = the cross-sectional area of the stirrup,
s = the spacing of stirrups,
Q = the total shearing force across the section,
P_{st} = the permissible tensile stress in the stirrups,
and l_a = the arm of the resistance moment.

If expression (1) is used to calculate the stress in the stirrups corresponding to the shearing force at failure of the beam, the resulting value can exceed, in some cases, the actual yield stress of the stirrups as many as seven times[3]. Measurement of the tensile stress in stirrups has shown that after the diagonal tension crack has opened, the stress in them increases with increasing load but the load on the beam can increase considerably beyond that causing the yield of the stirrups[1,3]. In this connection, Neville and Taub[3] suggested that the function of stirrups is not only to resist a part of the diagonal tension but also to restrain the downward deformation of the tension reinforcement of the beam and thus to restrain or even prevent horizontal splitting of concrete at the level of this reinforcement, and to preserve the integrity of the beam. This action of stirrups, although not easily expressed quantitatively, is believed to be of considerable importance.

It seems, therefore, that while the usual calculation of diagonal tension stresses is valid, and indeed useful, in the stage prior to the development of the diagonal tension crack, once this crack has widened a redistribution of internal stresses takes place. As a consequence, the further resistance of the beam to the action of shearing forces and its ultimate strength in shear depend on the strength of the compression and tension zones of the beam.

Types of Failure in Shear

It may be interesting at this stage to describe briefly the actual mode of failure of beams consequent upon the widening of the diagonal tension crack. There are four broad possibilities.

In the first, the upward extension of the diagonal tension crack can reduce the depth of the compression zone to such an extent that it cannot withstand the resultant compressive stress due to the bending moment and the shearing force applied, and failure by

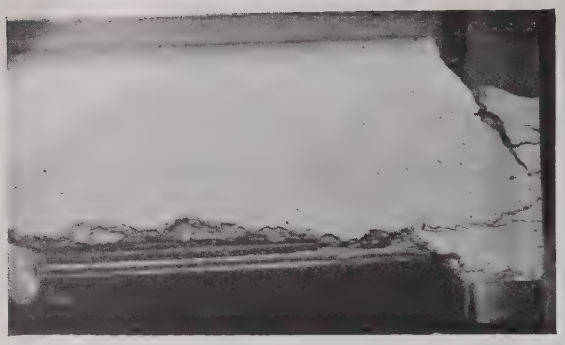


Fig. 1.—The tension steel in a beam after shear-tension failure

crushing occurs. This is known as shear-compression failure, and was first recognised by Laupa, Siess and Newmark[4]. A typical case of shear-compression failure is beam R74 of Fig. 9.

If, however, the compression zone is sufficiently strong to prevent crushing, as for instance is the case with T-beams, failure in shear-compression cannot take place. The diagonal tension crack can then extend horizontally at the level of the tension reinforcement toward the support. This situation arises when the concrete ceases to resist diagonal tension and allows the diagonal tension crack to widen. Vertical deformation of the tension reinforcement then takes place at the bottom end of the diagonal tension crack and a vertical shearing stress is induced in the tension steel[2]. An indication of this deformation can be obtained from Fig. 1. The shearing force in the tension steel, often referred to as dowel action, was actually measured by Jones[5]. More recently, Watstein and Mathey[6] observed the variation in the magnitude of this shearing force with an increase in the shearing force on the beam ; their data are in accord with the redistribution of internal forces postulated by Taub and Neville[2].

The extension of the horizontal crack results in the d struction of bond and consequently the tension in the main reinforcement is constant over the distance from the bottom of the diagonal tension crack to the end of the horizontal split (see Fig. 5b). This split gradually extends toward the support so that finally the tension steel slips or, if hooks are provided, the beam acts as a two-hinged tied arch. Such behaviour was first observed by Evans[7] as long ago as 1935. Taub[8] suggested the name of shear-tension for this mode of failure. A typical case is beam T65 of Fig. 7.

The strength of a beam in which shear-tension failure is developing clearly depends on the strength of the tension zone, namely on the amount of the tension steel in the vicinity of the supports, on the bond characteristics of this steel and on its anchorage. The influence of the first of these factors was demonstrated by Taub and Neville[2], while experiments on the other two are described in this paper.

If the anchorage of the tension steel is sufficiently strong the development of the shear-tension failure can be arrested and the ultimate failure may occur in another way, usually due to the yield of the tension steel in the zone of maximum bending moment. This then is the third mode of failure under the action of shear and bending moment. It is possible, however, only in a beam with web reinforcement as the support of the main reinforcement by stirrups is essential for the composite action of the beam to be preserved ; this argument has been mentioned earlier.

The fourth and last possibility is for the upper end of the diagonal tension crack to extend at a continuously decreasing slope until separation of the beam into two parts takes place. Examples of this type of failure are beams T66 and T67 of Fig. 7. It was first observed by Ferguson[9] who explained it in terms of the principal tensile stress in the compression zone of the beam. The magnitude of the flexure stresses increases towards the top surface of the beam and there is also some decrease in the shearing stress, so that the directions of the principal stresses become closer to horizontal and vertical. While this argument seems logical and the proportions of the beam and the $\frac{a}{d}$ ratio can be shown to influence the type of failure, it is not quite clear what determines the ' choice ' between a vertical tension and horizontal compression failure.

Details of Tests

The foregoing discussion has shown that the strength in shear of a reinforced concrete beam depends on a number of factors, in addition to those included in expression (1). Experimental data on the influence of some of these factors will now be given.

All the beams tested were of the dimensions shown in Fig. 2, three types of cross-section, rectangular, T- and L- being used. All the beams were simply supported over a span of 5 ft., and where web reinforcement was provided it consisted of $\frac{1}{4}$ inch diameter vertical stirrups in the shear span, spaced at $4\frac{1}{2}$ inch centres.

Three shear spans were used : 11, 16 and 23 inch, giving shear span-effective depth ratios of 1·63, 2·37 and 3·41 respectively.

The Size of the Compression Zone

The accepted method of calculation for shear recognizes no difference between the shear strengths of rectangular and T-beams of the same width below the neutral axis and, in fact, only a small increase in the load under which the diagonal tension crack has first formed in T-beams was observed. This was probably due to their higher second moment of area,

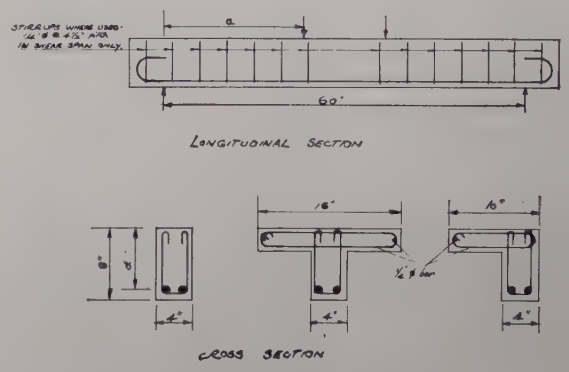


Fig. 2.—Details of beams tested

Table 1.—Test Data for Rectangular, T- and L- beams

Series	Type of Beam	Stirrups	$\frac{a}{d}$	p per cent	Compressive strength lb/sq.in.	Load under which diagonal tension crack developed ton	Ultimate load ton	Increase in ultimate load over that of rectangular beam per cent	Mode of failure
1	Rectangular	No	2·76	3·40	3680	4·1	5·6		Shear-compression.
	T				3480	4·9	8·9	59	Destruction of beam end.
2	Rectangular	No	2·91	3·60	4770	3·6	7·2		Destruction of beam end.
	T				4480	4·0	8·5	18	Destruction of beam end.
3	Rectangular	No	2·37	4·46	5950	3·9	8·7		Destruction of beam end.
	L				5120	5·0	9·0	3	Destruction of beam end.
	T				6680	7·4	9·9	14	Destruction of beam end.
4	Rectangular	No	3·41	4·46	4400	4·2	4·9		Flat slope separation.
	L				5500	4·0	6·4	31	Destruction of beam end.
	T				5200	5·0	7·9	61	Destruction of beam end.
5	Rectangular	Yes	2·37	4·46	6420	5·5	13·0		Destruction of beam end.
	L				5180	5·5	12·8	–1	Destruction of beam end.
	T				5720	7·0	13·9	7	Destruction of beam end.
6	Rectangular	Yes	3·41	4·46	5040	5·2	10·3		Flexure-compression.
	L				5120	4·5	9·9	–4	Flexure-tension.
	T				4990	5·0	12·6	22	Flexure-tension and incipient destruction of beam end.

Note : p is the percentage steel area ratio based on the web width.

and is in agreement with our argument about the ' elastic ' behaviour of reinforced concrete beams.

A comparison of ultimate loads, however, shows a considerable influence of the presence of the flange. The relevant test results are collected in Table 1.

For the beams without stirrups with the $\frac{a}{d}$ ratio of 2·37 (series 3) the ultimate loads are in the increasing order for the rectangular, L- and T-beams, while the central deflections of the same beams are, as would be expected, in the decreasing order. The deflection influences the degree of opening of the diagonal tension crack and the splitting along the main reinforcement. The values of the deflection of the beams of series 3 to 6 are plotted in Fig. 3.

All beams of series 3 failed in shear-tension by the destruction of the beam end because the low value of $\frac{a}{d}$ of this series leaves little room for the flat slope crack to develop.

Of the beams without stirrups but with the $\frac{a}{d}$ ratio of 3·41 (series 4) the rectangular beam failed by flat slope separation, while the L- and T-beams failed by end destruction (see Fig. 6). The ultimate loads were in the same order as for series 3, and it seems that for a larger value of the $\frac{a}{d}$ ratio the presence of the flange may be effective in preventing the separation type of failure which occurred in the rectangular beam.

Let us consider now beams with stirrups. In those with the $\frac{a}{d}$ ratio of 2·37 (series 5), there is not sufficient room for the development of a flat slope separation, and they all failed by end destruction, although the horizontal split was considerably restrained by the stirrups : it consists of a series of short cracks, as shown in Fig. 4. For this reason the difference in the ultimate loads of beams of series 5 is small : with the strong influence of stirrups the deflection (Fig. 3) is of lesser importance, and the stress in the tension reinforcement becomes the governing factor.

The beams with stirrups and the $\frac{a}{d}$ ratio of 3·41 (series 6) all failed in flexure, i.e., they were efficiently reinforced against shear. As would be expected, the rectangular beams failed in compression and the L- and T-beams by the yield of the tension reinforcement. All three beams, however, showed distinct diagonal tension cracks, and the T-beam also suffered partial end destruction.

It is clear then that the ultimate strength in shear of T-beams is generally higher than that of rectangular beams. In the present tests the increase has been found to range from 7 to 61 per cent of the strength of a rectangular beam of the same web width, the increase being greater the more favourable the conditions for shear failure. The increase in strength seems also to be greater when the beams deflect more, and may depend too on the type of failure actually taking place.

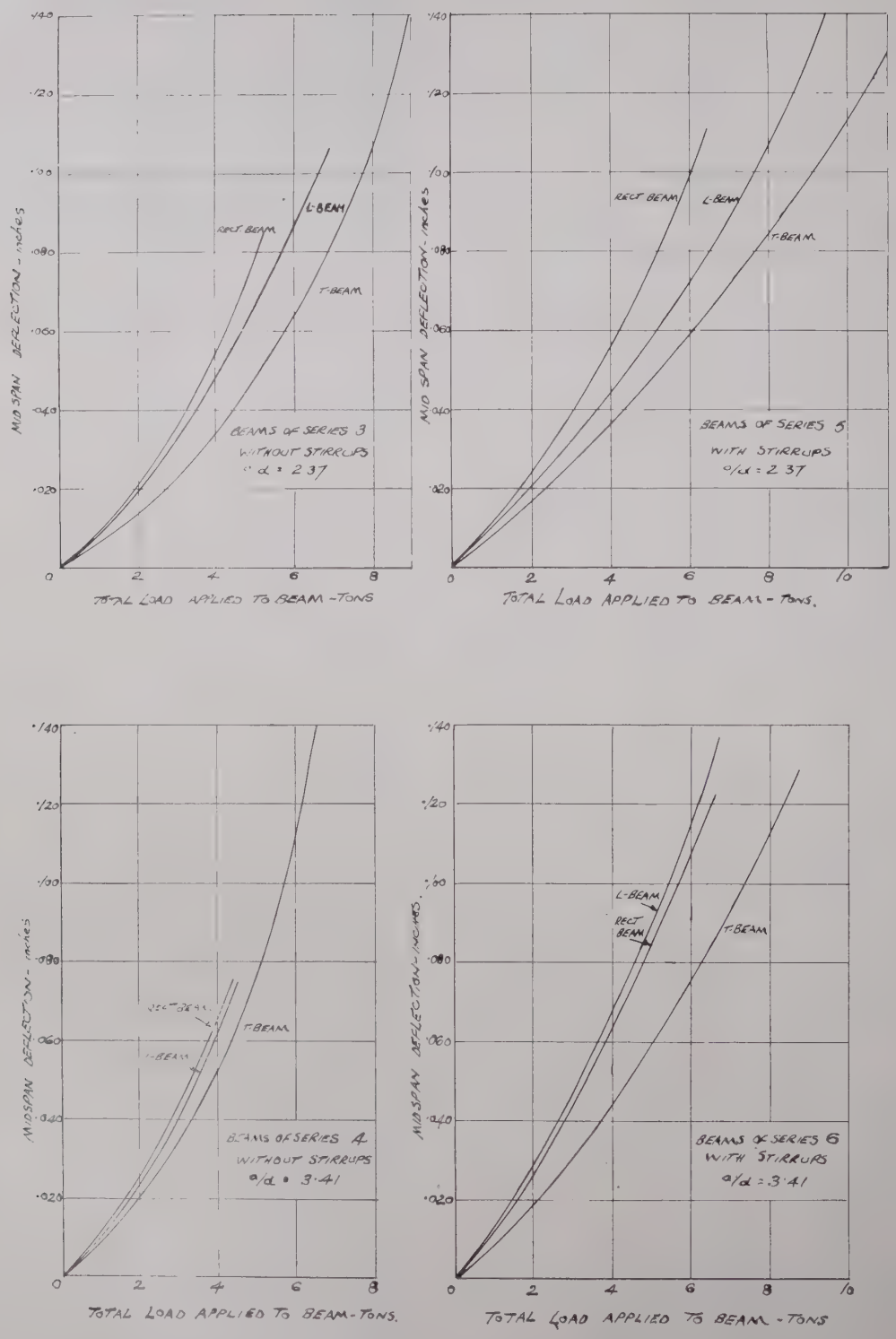


Fig. 3.—Deflection at mid-span of rectangular, T- and L-beams. (Table I)

Table 2.—Test Data for T-beams with Plain Reinforcement with and without Hooks

Beam Series	Stirrups	$\frac{a}{d}$	Hooks	Ultimate load ton.	Increase in ultimate load due to hooks. per cent.	Mode of failure
15	No	3·41	No	6·50		Slip of tension steel.
17			Yes	7·82	20	Destruction of beam end.
16	Yes	3·41	No	10·75		Slip of tension steel.
35			Yes	12·60	17	Flexure-tension and incipient destruction of beam end.

Table 3.—Test Data for T-beams with Deformed Reinforcement with and without Hooks

Beam Series	Stirrups	$\frac{a}{d}$	Hooks	Compressive strength lb/sq. in.	Ultimate load ton	Mode of failure
63	No	1·63	No	3330	13·4	Slip of tension steel.
64			Yes	3900	13·0	Flat slope separation.
65	No	2·37	No	4030	10·3	Slip of tension steel.
66			Yes	4300	10·8	Flat slope separation.
60	No	3·41	No	3660	6·2	Flat slope separation with some splitting at level of steel.
61			Yes	3940	6·2	Flat slope separation with incipient destruction of beam end.
68	Yes	1·63	No	5230	19·0	Flat slope separation.
67			Yes	4420	20·6	Flat slope separation.
90	Yes	2·37	No	4740	15·3	Flat slope separation with some splitting at level of steel.
69			Yes	3910	14·9	Flat slope separation.
72	Yes	3·41	No	4660	13·5	Multiple diagonal tension cracks with slip of tension steel.
71			Yes	4190	13·5	Multiple diagonal tension cracks with flat slope separation.

Table 4.—Test Data for T-beams with Different Tension Reinforcement

Beam Series	Number and diameter of tension bars	Type of reinforcement	p per cent	Sum of perimeters of bars. in.	Load under which horizontal splitting occurs. ton	Ultimate load ton	Mode of failure
17	2 —— $\frac{7}{8}$ in.	plain	4·46	5·50	6·0	7·9	Destruction of beam end.
19	3 —— $\frac{3}{4}$ in.	plain	4·94	7·08	5·0	6·0	Flat slope separation.
21	2 —— $\frac{3}{4}$ in. and 1 —— $\frac{5}{8}$ in.	plain	4·40	6·68	4·5	6·8	Destruction of beam end.
23	1 —— $1\frac{1}{4}$ in.	plain	4·54	3·93	5·0	9·4	Destruction of beam end.
13	2 —— $\frac{7}{8}$ in.	deformed	4·46	5·50 nominal	4·7	7·0	Flat slope separation.

Note :p is the percentage steel area ratio based on the web width.

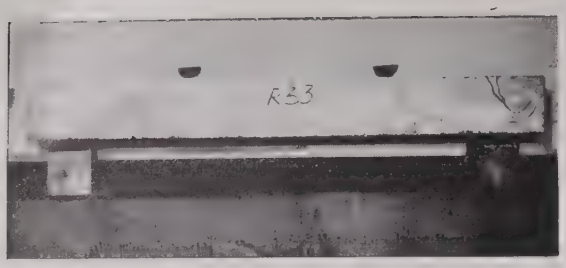


Fig. 4—Cracking at the level of the tension steel in a beam with vertical stirrups

Turning now to L-beams, we can see that beams without stirrups fail at loads intermediate between rectangular and T-beams. On the other hand, L-beams with stirrups are no stronger than the corresponding rectangular beams because the principal axes of an L-beam do not coincide with the axes of bending and loading and, under the heavy loads resisted by the beams with stirrups, severe cracking of the L-flange was found to take place. The cracks generally travelled round the edge of the flange and onto its top so that the effective beam section was no greater than that of a rectangular beam. As mentioned earlier, in L-beams without stirrups the loads acting were considerably lower so that this extensive cracking of the L-flange did not develop.

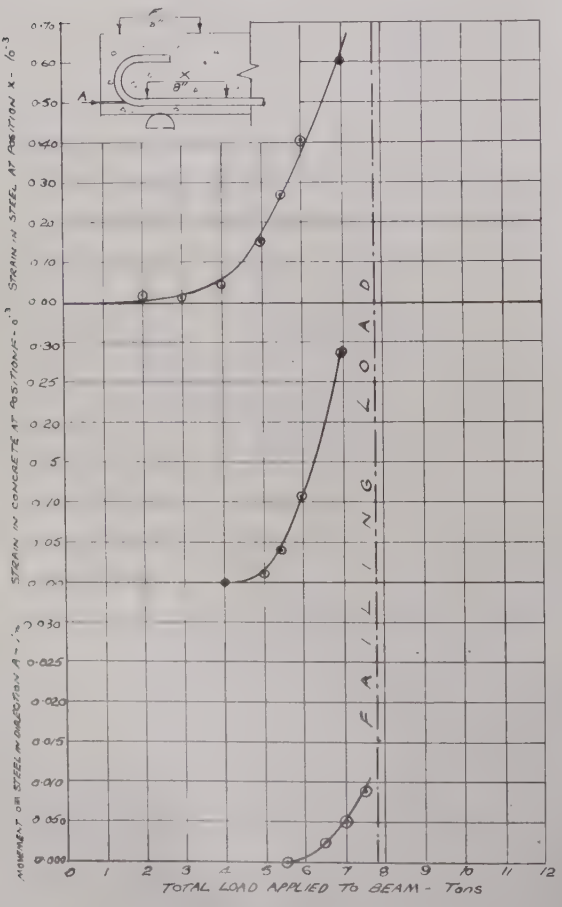


Fig. 5a—Strains and slip of reinforcement in T-beam of series 4 (Table I)

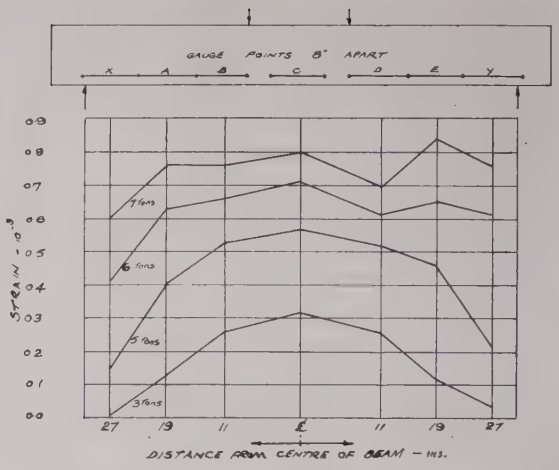


Fig. 5b—Strain distribution along reinforcement in T-beam of series 4 (Table I)

The Influence of Hooks on Plain Bars

The change from the beam action to a regime of forces existing in a two-hinged tied arch in a beam in which shear-tension failure has started, was mentioned before, and experimental data on the increase in the load carrying capacity of such beams due to the provision of hooks have been reported by Lewis[10] and by Taub and Neville[2]. Such an increase in the load-carrying capacity of beams is not surprising but is nevertheless of considerable interest in that it shows the influence of anchorage on the shear strength of a reinforced concrete beam. For this reason a summary of data on two pairs of T-beams, one with and the other without stirrups, is presented in Table 2. It can be seen that the presence of hooks results in an increase of some 20 per cent in the ultimate load on the beam. Fig. 5 shows a plot of the strain development in the tension reinforcement near the support and in the top surface of the concrete above, thus confirming the tied-arch action in the beam. The slip of reinforcement and the strain distribution in it are also shown.



Fig. 6—Destruction of a beam end by a hook

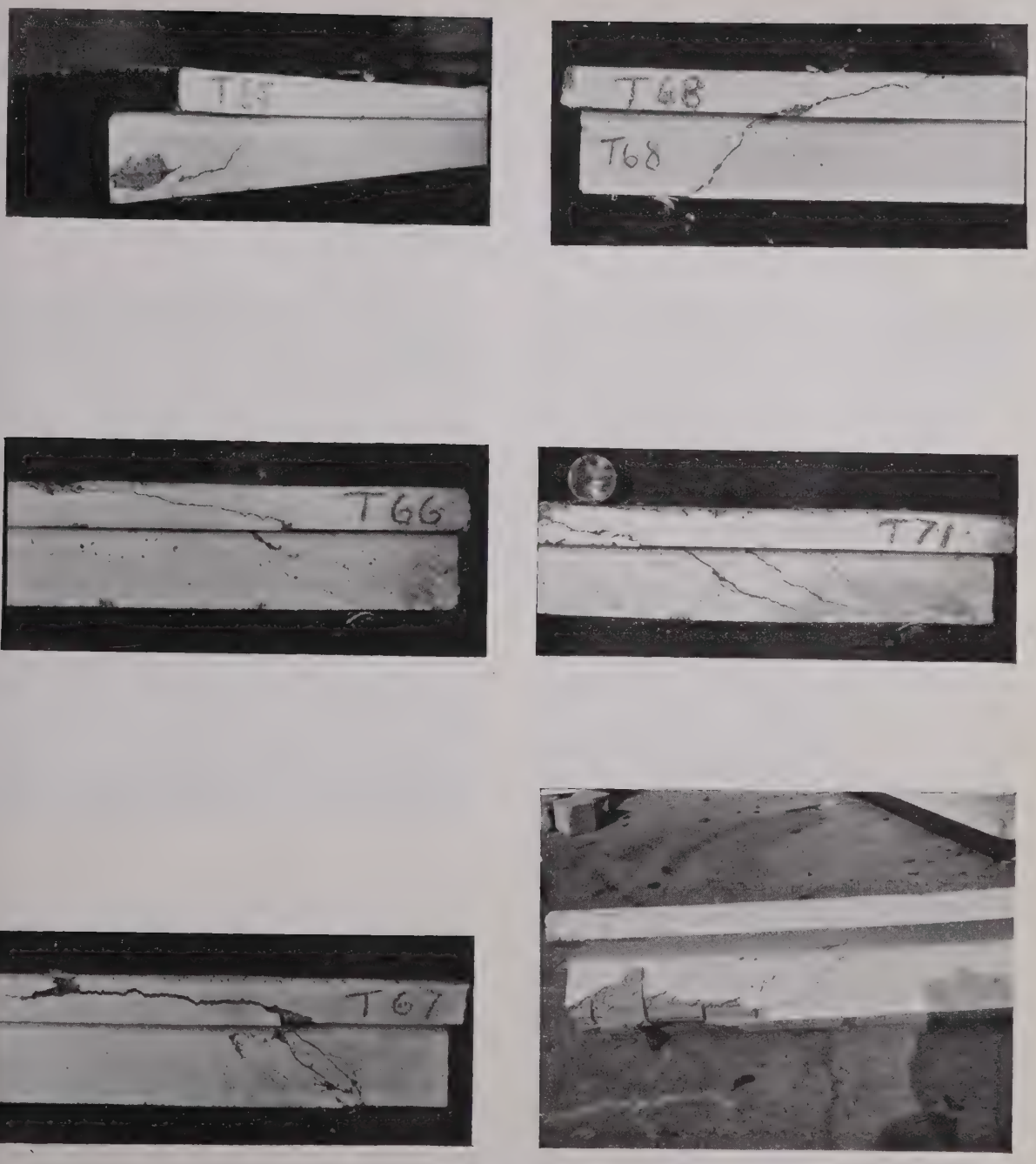


Fig. 7—Crack patterns at failure of some beams of Table 3

When the load exceeds that which can be resisted by the concrete surrounding the hooks splitting takes place. Such a destruction of the beam end by hooks is shown in Fig. 6.

Hooks on Deformed Bars

Deformed bars, because of their improved bond characteristics, are generally used without hooks. It was thought, however, that the possibility of the influence of hooks should be checked, and Table 3 summarizes average values of duplicate test results for a number of T-beams with Tentor reinforcement, arranged in pairs differing only by the presence or absence of hooks. Beams both with and without stirrups were tested, and three shear spans (11, 16 and 23 inches) were used for each type of beam.

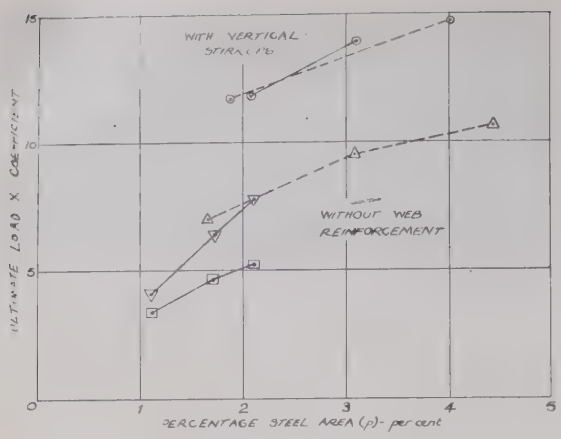


Fig. 8—The influence of the steel area ratio on the shear strength of reinforced concrete beams

Considering first beams without stirrups, it can be seen from Table 3 that the hooks do not affect the load carrying capacity of the beam but they influence its mode of failure. All beams with hooks failed by the flat slope separation, while of the beams without hooks, only the beam with the $\frac{a}{d}$ ratio of 3·41 failed in this manner. Typical crack patterns are shown in Fig. 7. For lower values of the $\frac{a}{d}$ ratio the beams without hooks failed due to the slip of the tension steel, in a manner similar to the failure of beams with plain round reinforcement. It would seem, therefore, that the presence of hooks prevents slip of the reinforcement from taking place, and failure must thus occur at the upper end of the diagonal tension crack. In the beams tested, either form of failure occurred under approximately the same load.

It is of interest to note that despite the higher bond strength of the deformed bars they can slip under the circumstances of shear failure when no stirrups are present. Such slip would be prevented by the addition of hooks.

When, however, stirrups are provided only incipient slip is possible, and all but one of the beams tested were observed to fail by the flat slope separation (Fig. 7). It would seem, therefore, that stirrups are valuable in preserving the integrity of beams with deformed tension steel. The stirrups themselves were, of course, made of mild steel.

Since the mode of failure of all the beams was the same, their strength depended on the magnitude of the principal tensile stresses in the flange, and was, therefore, unaffected by the hooks. The actual failing loads varied inversely with the length of the shear span.

The Tension Reinforcement

The increase in the shear strength of reinforced concrete beams with increase in the area of the tension reinforcement in the shear span has been reported earlier[2] but it may be interesting to show this relationship graphically. Fig. 8 presents the relevant data from various investigations both for beams with vertical stirrups and for beams without web reinforcement.

Furthermore, there are indications that, for a given cross-sectional area of tension reinforcement, the number of bars in one horizontal plane affects the load under which splitting at the level of this reinforcement takes place. It seems that the greater the sum of diameters of bars in one plane the lower the effective cross-section of concrete that can withstand the tendency to splitting induced by the shear in the tension steel, but further tests are required for this to be confirmed.

The difficulty of interpreting data arises from the fact that bond strength also affects the load under which horizontal splitting at the level of the tension steel occurs. This is indicated by the values of this load in Table 4, but it is clear that the data are not conclusive.

That the extent of the further development of the horizontal cracks at the level of the tension steel and the ultimate load of the beam depend on the degree of opening of the diagonal tension crack and not merely on bond stresses is shown by the values of the ultimate load in Table 4.

All the beams were of T-shape, without stirrups, had tension reinforcement with hooks, and were tested with the $\frac{a}{d}$ ratio of 3·41. The beam reinforced by one $1\frac{1}{4}$ inch diameter bar (No. 23) had the highest ultimate strength, probably because a bar of that size can better resist the deformation at the bottom end of the diagonal tension crack. This is in agreement with the theoretical argument of Taub and Neville[2].

The high strength of beam No. 23 may also be due, in part, to the fact that the hooks at the end of the $1\frac{1}{4}$ inch diameter bar, being of standard size, had their top parts embedded in the flange of the beam: this could offer a greater resistance to destruction than is the case when the hooks are wholly in the web of the beam.

A comparison of beams Nos. 19 and 23 shows also the importance of the position of the main diagonal tension crack. This, as any other form of cracking in concrete, is influenced by the presence of flaws in the material and the consequent variation in the strength of concrete within the shear span[11]. When the diagonal tension crack is in such a position that a flat slope separation can occur this may result in a lower ultimate load than when failure occurs due to horizontal splitting and destruction of the beam end. This is the case in the beams considered but need not be so in beams of other proportions.

Beam No. 13 with Tentor bars was no stronger than beams with plain reinforcement, possibly again because failure occurred by a flat slope separation. On the other hand, it seems that when the split along the tension steel develops to a considerable degree the deformation of the beam is such that failure by separation cannot take place and the strength of the beam is that of the resultant 'tied arch.'

Beams with Compression Reinforcement

It was shown earlier that the addition of flanges to a rectangular reinforced concrete beam increases its resistance to shear, and as a further step it would be interesting to know also the influence of compression reinforcement.

The existing experimental data on this subject are not very clear. Wilby[12] found the load carrying capacity in shear to decrease with an increase in the percentage area of compression reinforcement, in the case of rectangular beams with $\frac{a}{d}$-ratios between 2·1 and 3·2.

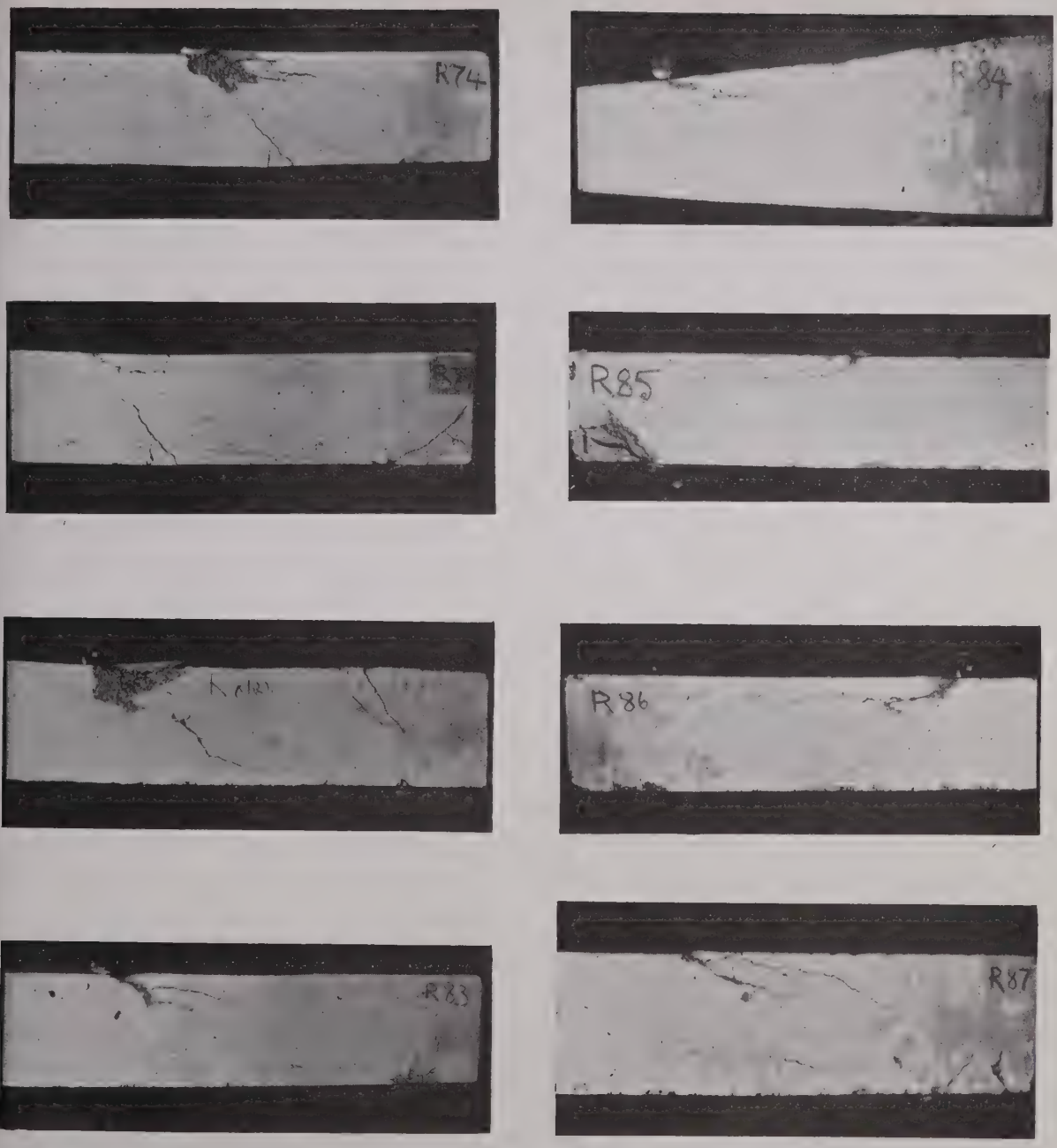


Fig. 9—Crack patterns at failure of some beams of Table 5

On the other hand, the strength of his T-beams, with $\frac{a}{d}$ ratios between 2·1 and 6·2, was not affected by the presence or amount of the compression reinforcement. Likewise, Bernander's[13] tests show no effect of the compression reinforcement on the shear strength of rectangular beams with high tensile reinforcement and $\frac{a}{d}$ ratio of 4. This would also appear to be the case in Al-Alusi's[14] tests on T-beams and inverted T-beams without web reinforcement.

On the other hand, Laupa, Siess and Newmark's[4] analysis shows the compression reinforcement to contribute to the shear strength of rectangular beams in proportion to the amount of this reinforcement. Jones'[5] tests on beams with inclined stirrups and a pre-formed diagonal tension crack indicate that the compression reinforcement carries a considerable proportion of vertical shear on the beam.

Table 5.—Test Data for Rectangular Beams with Different Amounts of Compression Reinforcement

Beam Series	Shear span in.	Number and diameter of compression bars	Cross-sectional area of compression reinforcement sq. in.	Compressive strength lb/sq. in.	Ultimate load ton	Mode of failure
76	16	none	0·000	4370	12·0	Shear-compression.
74		2 — $\frac{1}{4}$ in.	0·098	4130	10·9	Shear-compression.
75		1 — $\frac{1}{2}$ in.	0·196	3780	9·9	Shear-compression.
73		2 — $\frac{3}{8}$ in.	0·221	3870	10·4	Shear-compression.
87		2 — $\frac{3}{4}$ in.	0·884	3800	10·6	Destruction of beam end.
85		2 — 1 in.	1·571	4430	11·2	Destruction of beam end.
77		2 — 1 in. and 1 — $\frac{7}{8}$ in.	2·172	3200	12·4	Destruction of beam end.
78		2 — $1\frac{1}{4}$ in.	2·450	4480	11·3	Destruction of beam end.
82	23	none	0·000	4430	10·2	Flexure-compression.
80		2 — $\frac{1}{4}$ in.	0·098	4700	9·8	Shear-compression.
81		1 — $\frac{1}{2}$ in.	0·196	4380	9·0	Shear-compression.
79		2 — $\frac{3}{8}$ in.	0·221	3820	10·6	Shear-compression.
5		2 — $\frac{1}{2}$ in.	0·393	4190	9·8	Shear-compression.
9		2 — $\frac{5}{8}$ in.	0·614	4240	9·5	Shear-compression.
88		2 — $\frac{3}{4}$ in.	0·884	4600	9·4	Shear-compression.
86		2 — 1 in.	1·571	4600	9·3	Flat slope separation with incipient shear compression.
83		2 — 1 in. and 1 — $\frac{7}{8}$ in.	2·172	4780	10·0	Destruction of beam end.
84		2 — $1\frac{1}{4}$ in.	2·450	4610	9·4	Destruction of beam end.

The existing data seem thus to be conflicting and for this reason some new tests on beams with varying amounts of compression reinforcement are reported. All the beams tested had vertical stirrups and tension reinforcement consisting of two $\frac{7}{8}$ in. diameter plain bars with hooks. Two shear spans, 16 and 23 inches were used, corresponding to $\frac{a}{d}$ ratios of approximately 2·4 and 3·4 respectively. The test data are collected in Table 5.

Let us consider first the beams with the shear span of 16 inches. The beam without compression steel and those with up to about 0·22 sq. in. of compression reinforcement failed in shear-compression. In the beam with 0·88 sq. in. of compression steel considerable cracking along this reinforcement is apparent (Fig. 9), and the failure of the beam was due to destruction of its end by the hooks. With an increasing amount of compression steel up to 2·45 sq. in. the cracking at the level of that steel becomes progressively more severe but all these beams failed due to the destruction of their ends by the hooks.

The cracking in the vicinity of the compression reinforcement is probably due to the reduced width of concrete there able to withstand principal tensile stresses, the resistance to pull of the steel-concrete interface being lower than the tensile strength of concrete.

Turning now to the beams with a shear span of 23 in. we see that the beam without compression steel failed in flexure-compression. The beams with up to 0·88 sq. in. of compression reinforcement failed in shear-compression, without any cracks developing at the level of the compression steel. For 1·57 sq. in. of compression reinforcement cracking at this level occurred and failure took place by the flat slope separation. For higher amounts of compression steel severe cracking at this level was apparent and failure was due to the destruction of the beam end (Fig. 9).

It seems then that the changes in the crack and failure patterns for the two shear spans are similar but occur for higher amounts of compression steel in the case of the 23 inch shear span than for the 16 inch shear span. This is, of course, so because the larger the shear span the larger the flexure stresses for the same load on the beam.

Although cracking is influenced by the compression steel it seems that the failing load varies only little, possibly because the ultimate failure occurs perforce away from this steel so that the stress distribution immediately prior to collapse is unaffected.

Further tests may give a better understanding of the failure of beams with compression reinforcement but the present investigation is considered of interest in establishing that the shear strength of rectangular beams with vertical stirrups is practically independent of the amount of the compression reinforcement.

Conclusions

The comparison of rectangular and T-beams has shown that the size of the compression zone affects the strength of a beam in shear—a point of interest with regard to design. This could explain why the rectangular beams in some U.S. Air Force warehouses failed in shear while the beams cast integrally with slabs (i.e. effective T-beams) remained intact[15].

The study of beams with and without hooks indicates that the tension zone too is a factor in the shear strength of a beam. This points also to a close connection between shear and anchorage. On the other hand, although it affects crack patterns and the mode of failure, compression reinforcement appears not to affect the shear strength of beams of the type tested. Further tests are required to check whether this is generally true. Nevertheless, the data reported are of interest because the amount of compression reinforcement has in the past been included in empirical formulae for shear strength[4].

Acknowledgements

The Authors are grateful to Professor J. A. L. Matheson of the University of Manchester for enabling them to work on the problem of shear. They are also pleased to acknowledge that their interest in some of the problems investigated arose from discussion with Dr. J. Taub, with whom one of the authors (A. M. N.) has conducted a more general study of the problem of shear.

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Discussion

The Council would be glad to consider the publication of correspondence in connection with the above paper. Communications on this subject intended for publication should be forwarded to reach the Institution by the 30th September, 1960.

Book Review

Strength of Materials for Engineers, by D. R. Axelrad. (London : Pitmans, 1959). 8vo. 546 pp., 55s.

The volume is intended to meet the requirements of students preparing for Engineering Degree examinations and for the final examinations of Professional Institutions.

The author has succeeded in bringing together in one volume of about 550 pages a comprehensive treatment of the subject in a way which will appeal to most students.

The subject matter is divided into three main sections :—

(1) Graphical and Analytical Statics.
(2) Mechanical Properties and Testing of Materials.
(3) Strength of Materials.

As would be expected in a book of this nature, much that is already well accepted is contained and is treated along quite conventional lines, but the careful planning which has obviously taken place renders a logical sequence to this material and leads in the writer's opinion to a sensible approach.

The book is well illustrated throughout and the chapters are covered by worked examples of a conventional nature.

The chapter devoted to the analysis of forces in frames could have been rather more lengthy, and more examples here would have been appreciated.

An excellent treatment of energy principles forms the subject of a complete chapter however, though here again it is felt that a little more space could have been given to the treatment of statically indeterminate structures.

The book nevertheless covers in some detail all that is claimed of it, and will be a welcome addition to the bookshelves of senior students in the Technical Colleges and others working for their engineering degrees.

D.T.W.

Deflections of Plane Frames at the Point of Collapse

by Professor B. G. Neal, M.A., Ph.D., .M.I.C.E.

Summary

The flexibility which is inherent in nominally rigid joints and footings in plane frames is known not to affect the value of the plastic collapse load. The object of the present paper is to show that substantial amounts of flexibility can often occur without affecting the deflections at the point of collapse, that is when the plastic collapse load has just been attained but no motion of the collapse mechanism has taken place. This is in striking contrast to the obvious increase in the deflections in the elastic range of behaviour when flexibility of any kind is present.

After illustrating the general principles involved by referring to a simple beam problem, a rectangular portal frame is analysed to show how a lower limit for any particular joint or base stiffness can be determined, above which the deflections at the point of collapse are unaffected.

Introduction

In elastic analyses of framed structures, it is usually assumed that no deflections occur at column bases, and that a base which is designed to resist bending will not undergo any rotation. Welded joints are also assumed to be fully rigid, so that no relative rotation of members will occur. These assumptions are known to be unrealistic, for there is inevitably some degree of flexibility in any footing or joint. This inherent flexibility influences the elastic distribution of stress, and also lowers the stiffness of the frame so that deflections are increased.

The plastic methods of structural design for framed structures are based on the prediction of the load at which a framed structure will fail by plastic collapse, that is, by forming enough plastic hinges to reduce the structure to a mechanism. It is well known that the value of the plastic collapse load for a given frame and loading does not depend on the degree of flexibility in the footings and joints. Nevertheless, it might well be expected that flexibility of any kind would increase the deflections at the point of collapse, that is when the plastic collapse load has been attained but no motion of the collapse mechanism has yet occurred. However, the purpose of this paper is to show that in many cases a substantial degree of flexibility can be present while leaving the deflections at the point of collapse unchanged at the values for a fully rigid frame. A simple technique is also presented for determining the critical value of a joint or base stiffness of any kind above which the deflections at the point of collapse are unaffected.

Methods are already available for estimating the deflections of plane frames at the point of collapse[1,2]. In forming these estimates, certain simplifying assumptions are made regarding the behaviour of the structure in the plastic range. In the first place, it is assumed that the effect of the spread of plastic zones along a member from the point of location of a plastic hinge can be neglected, so that the members are supposed to remain entirely elastic except at the plastic hinge positions. Secondly, the effect of strain-hardening at the plastic hinges is neglected. Because of the first assumption the deflections are underestimated, but the effect of strain-hardening is to reduce deflections, and it has therefore been suggested that when the two assumptions are taken together the resulting estimates of deflection will be quite accurate. This suggestion gains credence from a number of comparisons with test results which have been made[2].

A further assumption which must be made is that the loading programme is such that once a plastic hinge has formed it will always continue to rotate as the loads change. This implies that as the loads increase, all the plastic hinges involved in the collapse mechanism will form and rotate in turn, and no other plastic hinges will be formed at any stage. This will generally be true under proportional loading, in which all the load components are proportional to one another, and will also be true in some other cases. It is obviously untrue for variable repeated loading of the kind that can cause incremental collapse[3].

A simple illustration of the effect of flexibility on the value of the deflections at the point of collapse is afforded by the case of a beam built in at both ends and carrying a uniformly distributed load. This example is first discussed in order to illustrate the principles involved. A simple rectangular portal frame is then analysed to show how deflection estimates would be found in a practical case, and how estimates could be made of the degrees of flexibility which could occur at the column bases and joints without affecting the deflections at the point of collapse.

Beam Built in at Both Ends

Consider first the uniform beam AB of length L, flexural rigidity EI and fully plastic moment M_p, which is built in at both ends and carries a uniformly distributed load W, as shown in Fig. 1. It will first

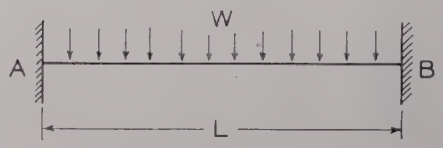


Fig 1.—Built in beam.

be assumed that the ends A and B always remain at the same level, but that when a bending moment M is developed at either end of the beam, a rotation θ occurs, such that

$$M = k\left(\frac{EI}{L}\right)\theta,$$

so that the non-dimensional parameter k defines the rotational flexibility of the end supports. When $k = \infty$, the ends are completely restrained against rotation, whereas when $k = 0$ the ends are simply supported.

Because of the symmetry of the beam and loading, local maximum values of the bending moment can only occur at the ends and centre of the beam, so that plastic hinges can only form at these three positions. A full analysis of the behaviour of the beam under a steadily increasing load can, therefore, be made by a simple step-by-step procedure. For example, if $k = \infty$, so that the ends of the beam are clamped, it is known that in the elastic range the hogging bending moment at each end of the beam is twice as great as the central sagging bending moment, so that a pair of plastic hinges will first form at the ends of the beam when the central sagging moment is $0{\cdot}5\,M_p$. As the load increases further, the end moments remain constant at the value M_p while the remainder of the beam stays elastic, until finally the central moment also becomes equal to M_p and collapse occurs. In this latter phase, the changes of bending moment and deflection are related to the change in load in exactly the same way as for an elastic simply supported beam, and can, therefore, be computed from known solutions.

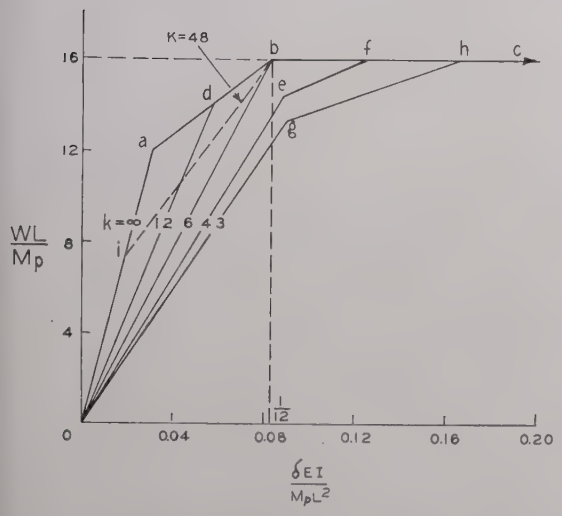


Fig. 2.—Load-deflection relations for built-in beam.

Load-deflection relations, calculated in this way for various values of k, are shown in Fig. 2, in which the ordinates represent the non-dimensional load parameter WL/M_p and the abscissae represent the non-dimensional deflection parameter $\delta EI/M_pL^2$. For the fully clamped beam, $k = \infty$, the relation is *Oabc*, the collapse load $W_c = 16M_p/L$ being attained at b, where the corresponding deflection δ_c at the point of collapse is $M_pL^2/12EI$. For $k = 12$ the relation is *Odbc*, plastic hinges forming at the ends of the beam at d. For $k = 6$ the relation is *Obc*, plastic hinges forming simultaneously at the ends and at the centre of the beam at b. The behaviour for any value of k between 6 and ∞ is similar to that shown for $k = 12$, differing only in the slope of the elastic line.

The remarkable feature of these relations is that the deflection δ_c at the point of collapse b is unaffected by the value of k, so long as k lies between 6 and ∞, although the deflections produced by a given load in the elastic range differ markedly. In fact, for $k = 6$ the deflection for a given load in the elastic range is exactly twice the deflection for $k = \infty$. This result is however, no longer true if k is less than 6. For instance, if $k = 4$, a hinge forms first at the centre of the beam at the point e, and the point of collapse is f, and for $k = 3$ the load-deflection relation is *Oghc*. This suggests that δ_c is only independent of the value of k so long as the central plastic hinge is the last one to form. This is in fact so, and the reason can be explained by considering the direct calculation of δ_c, first for the rigidly built-in case and then for finite values of k.

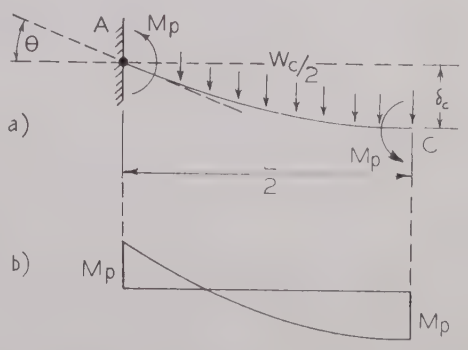


Fig. 3.—Rigidly built-in beam at point of collapse.

Direct Calculation of δ_c for Ends Rigidly Built in

Fig. 3(a) shows the deflected form of one half of the beam at the point of collapse, and the corresponding distribution of bending moment is shown in Fig. 3(b). The plastic hinge at the end A has rotated through some undetermined angle θ, but at the centre C the fully plastic moment has only just been attained, so that no plastic hinge rotation has occurred, and the slope of the beam is therefore zero. The total load on this half of the beam is $W_c/2$, or $8M_p/L$. Writing the two slope-deflection relations for this segment,

$$\theta = \frac{2\delta_c}{L} + \frac{L}{12EI}\left[2\left(-M_p + \frac{W_cL}{48}\right) - \left(-M_p - \frac{W_cL}{48}\right)\right] \quad . \quad (1)$$

$$0 = \frac{2\delta_c}{L} + \frac{L}{12EI}\left[2\left(-M_p - \frac{W_cL}{48}\right) - \left(-M_p + \frac{W_cL}{48}\right)\right] \quad . \quad (2)$$

and the solution of these equations is

$$\delta_c = \frac{M_pL^2}{12EI} \quad , \quad \theta = \frac{M_pL}{6EI} \quad . \quad . \quad . \quad (3)$$

It was tacitly assumed that the last hinge to form was at mid-span, so that at the point of collapse this hinge would not have undergone any rotation and the slope would be continuous there. It was then

found that the value of θ was positive, and from Fig. 3(a) it is seen that this is consistent with the sense of the hogging fully plastic moment at A. This verifies the assumption that the last hinge forms at C, so that the value of δ_c is as given in equation (3); had θ been negative it would have been incorrect to assume that the last hinge formed at C, and a fresh calculation assuming that the last hinges to form were at A and B would have been made.

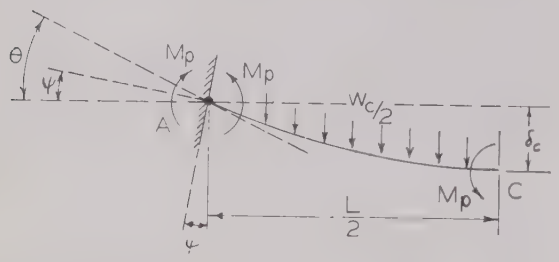


Fig. 4.—Flexibly restrained beam at point of collapse.

Effect of Rotational Flexibility

Consider now the situation when the ends of the beam are elastically restrained against rotation, $k < \infty$, as illustrated in Fig. 4. Since the beam is in a collapse condition, the bending moment distribution is completely determined from statical considerations, and is identical with the distribution shown in Fig. 3(b) for the rigidly built-in case. On the assumptions made, the portion AC is entirely elastic, so that the curvature distribution, and hence the curved shape, is also identical with that shown in Fig. 3(a). The values of δ_c and θ are therefore the same as for the clamped ended case, equations (3). However, the rotation of the plastic hinge at A is now $(\theta - \psi)$, where ψ is the rotation of the end support under the influence of the hogging bending moment M_p, and is given by

$$\psi = \frac{M_p L}{kEI} \quad \cdot \quad \cdot \quad \cdot \quad (4)$$

The plastic hinge rotation at A must be positive, for otherwise the sense of the hinge rotation would correspond to a sagging rather than a hogging fully plastic moment. The situation depicted in Fig. 4 is therefore only possible if

$$\psi < \theta$$

$$\text{or} \quad \frac{M_p L}{kEI} < \frac{M_p L}{6EI},$$

$$\text{so that} \quad k > 6,$$

using equations (3) and (4). If this condition is not fulfilled, the deformations at collapse which are indicated in Fig. 4 would not be possible, and this would imply that a plastic hinge would form first at the centre of the beam rather than at the ends. The deflection at the point of collapse would thereby be increased.

Sinking of Support

These ideas are readily extended to meet situations in which one or more of the supports sinks under load. Suppose for example that the end supports do not rotate, but the left hand support A sinks relative to the other support by an amount d which is proportional to the reaction R, so that

$$R = K\left(\frac{EI}{L^3}\right) d$$

If it is assumed that the last hinge to form is still at mid-span, the situation at the point of collapse will be as shown in Fig. 5. The curved shape of the beam will be unaltered, so that the deflections and slopes relative to the line AB will be unchanged. In particular, the central deflection δ_c relative to AB, and the slope $M_pL/6EI$ at each end relative to AB, will be the same. This situation is possible provided that the plastic hinge rotations at A and B are still in conformity with the hogging fully plastic moments at these sections. It will be seen from Fig. 5 that the effect of the sinking of A is to increase the hinge rotation at B, but the hinge rotation at A is reduced to $\left(\frac{M_pL}{6EI} - \frac{d}{L}\right)$

Hence the condition for the deflection relative to AB to remain unaltered is

$$\frac{M_p L}{6EI} > \frac{d}{L}$$

The value of R is found by taking moments about B to be $8M_p/L$. Putting $d = RL^3/KEI$, the above condition becomes

$$\frac{M_p L}{6EI} > \frac{8M_p L}{KEI}$$

$$\text{or} \quad K > 48$$

Thus if the value of K is not less than 48, the central deflection at the point of collapse, measured relative to the line joining the supports, remains unchanged.

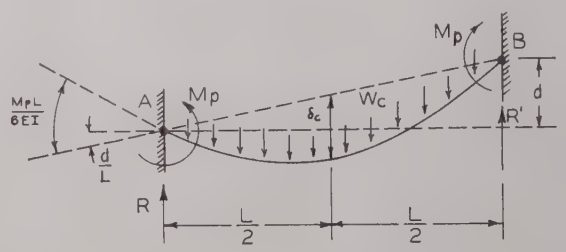


Fig. 5.—Effect of sinking of one support.

For reference, the load-deflection relation for the critical case in which $K = 48$ is given in Fig. 2 as 0*ibc*. Step-by-step calculations show that if K is less than this value, the last plastic hinge is formed at the support A, and of course the presence of a plastic hinge at mid-span which had already undergone rotation at the point of collapse would invalidate the above argument.

Rectangular Portal Frame

Fully Rigid Deflection Analysis

Consider now the rectangular portal frame whose dimensions and loading are as shown in Fig. 6(a). All the members of this frame are of uniform cross-section, with flexural rigidity EI and fully plastic moment M_p, and the feet of the columns are in the first place supposed to be rigidly built in. An estimate will first be made of the horizontal deflection h_c at beam level when the frame is at the point of collapse.

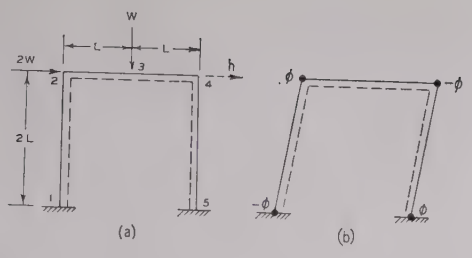


Fig. 6.—Rectangular portal frame : loading and collapse mechanism.

The situations which arise when relative sinking or spreading apart of the column bases occurs, or when rotation takes place at the column bases or at the joints, are then examined, and it is shown that values for the limiting degrees of flexibility beyond which the deflections at the point of collapse are increased can be found very readily.

A simple plastic analysis shows that the value W_c of W at which plastic collapse occurs is M_p/L, the collapse mechanism being the sidesway mechanism shown in Fig. 6(b). With the sign convention that a bending moment is regarded as positive if causing tension in the fibres of the member adjacent to the dotted line in Fig. 6(a), the bending moment distribution at collapse is as given below in Table 1.

Table 1

Bending Moment Distribution at Collapse

Cross section	1	2	3	4	5
Bending moment	$-M_p$	M_p	$\frac{1}{2}M_p$	$-M_p$	M_p

In order to begin the analysis for the determination of h_c, it is necessary to make an assumption (not necessarily correct) as to the location of the last plastic hinge to form. The arbitrary choice will be made that the last plastic hinge to form occurs at section 1, so that at the point of collapse it will first be assumed that there is no plastic hinge rotation at this section.

With this assumption, the deflected forms of the two columns and the beam are as shown in Fig. 7.

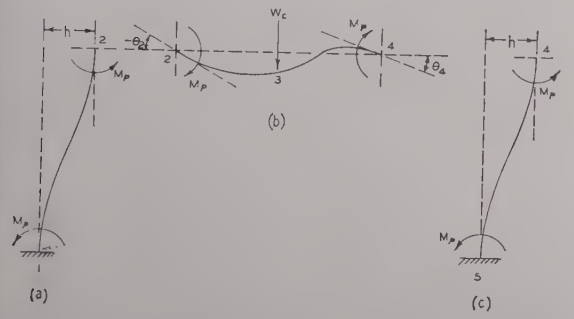


Fig. 7.—Initial assumptions for deflection analysis.

From the distribution of bending moment at collapse it is known that the columns are both bent in double curvature, so that the tangent at section 2 in Fig. 7 (a) is vertical. Since the horizontal deflection h at the top of each column must be the same, and their curved shapes are identical, it follows that the tangents at sections 4 and 5 in Fig. 7(c) are also vertical, and that the deflected forms of the two columns are the same. There is therefore only one slope-deflection relation for both the columns, namely

$$0 = \frac{h}{2L} + \frac{2L}{6EI}(-2M_p + M_p)$$

giving

$$h = \frac{2M_pL^2}{3EI} \quad . \quad . \quad . \quad (5)$$

In Fig. 7(b) the slope angles at the ends of the beam are equal in magnitude to the plastic hinge rotations at these sections, since the ends of the columns do not rotate. The signs of these hinge rotations have been made consistent with the sign convention for bending moments, so that a positive hinge rotation corresponds to tension in the fibres adjacent to the dotted line in Fig. 6(a). The two slope-deflection equations for the beam are

$$-\theta_2 = \frac{2L}{6EI}\left[2\left(M_p + \frac{W_cL}{4}\right) - \left(M_p - \frac{W_cL}{4}\right)\right]$$

$$\theta_4 = \frac{2L}{6EI}\left[2\left(M_p - \frac{W_cL}{4}\right) - \left(M_p + \frac{W_cL}{4}\right)\right]$$

whence

$$\theta_2 = -\frac{7M_pL}{12EI} \quad . \quad . \quad . \quad (6)$$

$$\theta_4 = \frac{M_pL}{12EI} \quad . \quad . \quad . \quad (7)$$

In fact, as will be seen from Table 1, θ_2 should be positive and θ_4 negative to be consistent with the signs of the fully plastic moments at these sections. This is achieved by adding to the deflections and hinge rotations just determined, the deflections and hinge rotations corresponding to a motion of the collapse mechanism of Fig. 6(b), which are geometrically compatible with zero changes of curvature in the members and do not therefore affect the curved *shapes* of the members depicted in Fig. 7. The appropriate value of φ in Fig. 6(b) is $\frac{7M_pL}{12EI}$, for with this value θ_4 is made negative and therefore consistent with the negative fully plastic moment at this section, while θ_2 becomes zero, indicating that the last hinge to form is at section 2. The calculations are summarized below in Table 2.

Table 2

Hinge rotations and deflections at point of collapse

Original Solution

θ_1	θ_2	θ_4	θ_5	h
0	$-\frac{7M_pL}{12EI}$	$\frac{M_pL}{12EI}$	0	$\frac{2M_pL^2}{3EI}$

Mechanism

θ_1	θ_2	θ_4	θ_5	h
$-\frac{7M_pL}{12EI}$	$\frac{7M_pL}{12EI}$	$-\frac{7M_pL}{12EI}$	$\frac{7M_pL}{12EI}$	$\frac{7M_pL^2}{6EI}$

Final Solution

θ_1	θ_2	θ_4	θ_5	h
$-\frac{7M_pL}{12EI}$	0	$-\frac{M_pL}{2EI}$	$\frac{7M_pL}{12EI}$	$\frac{11M_pL^2}{6EI}$

The deflected form of the frame at the point of collapse is indicated in Fig. 8.

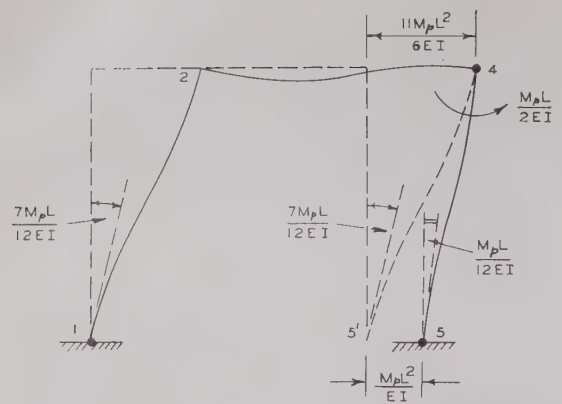


Fig. 8.—Deflected form at point of collapse.

Rotation at Column Bases

Consider now the situation which would arise if the column bases were elastically restrained against rotation in such a way that the application of a moment M caused a rotation θ, where $M = k\left(\frac{EI}{L}\right)\theta$. In the collapse situation just analysed, in which rigid bases were assumed, the plastic hinge rotation at each base was of magnitude $7M_pL/12EI$. At collapse, each base is subjected to the fully plastic moment and would therefore rotate clockwise through an angle M_pL/kEI. Assuming the deflected form of the frame to be unaffected, this would reduce the magnitude of the plastic hinge rotation at each base to $(7M_pL/12EI - M_pL/kEI)$. So long as the new plastic hinge rotations are still of the same sign as the fully plastic moments, the deflection analysis remains unchanged, so that the deflection at the point of collapse is unaffected. The condition for the deflections at the point of collapse to remain the same is therefore

$$\frac{7M_pL}{12EI} > \frac{M_pL}{kEI}$$

or $$k < \frac{12}{7}$$

For reference, the load-deflection relations for the fully rigid frame and for $k = 12/7$ are plotted in Fig. 9, in which the sections at which plastic hinges form are indicated at the salient points. It will be noticed that for the critical value of k, hinges do not form at the feet of the columns until the plastic collapse load is reached. This could have been foreseen from the above analysis, for when $k = 12/7$ the rotations of the members at the column bases at the point of collapse are exactly the same as the rotations of the bases themselves. The plastic hinge rotations are therefore zero at the point of collapse, so that hinges can only form at the column bases just as the plastic collapse load is attained.

Rotational Flexibility of Joints

The basis of the deflection calculations has been that there is complete continuity at section 2, where the last plastic hinge forms. Thus if this joint did not remain rectangular under load, the deflections would inevitably alter. However, at section 4, the plastic hinge rotation at the point of collapse is of magnitude $M_pL/2EI$. It follows that some degree of rotational flexibility can occur at this joint while leaving the deflections unchanged. The limiting case occurs when under the action of a moment M_p, the angle between the two members meeting at this joint changes by an amount $M_pL/2EI$ due to its rotational flexibility. The configuration of the frame at collapse would then be precisely the same as in Fig. 8, but the rotation at section 4 would then be due to rotational flexibility rather than to plastic hinge rotation. If the stiffness of the joint is defined by the non-dimensional parameter k^*, such that a moment M acting at the joint causes a relative rotation of the members θ given by $M = k^*\left(\frac{EI}{L}\right)\theta$, it follows that if

$$\frac{M_pL}{k^*EI} < \frac{M_pL}{2EI},$$

or $$k^* > 2,$$

the deflection at the point of collapse will remain unchanged.

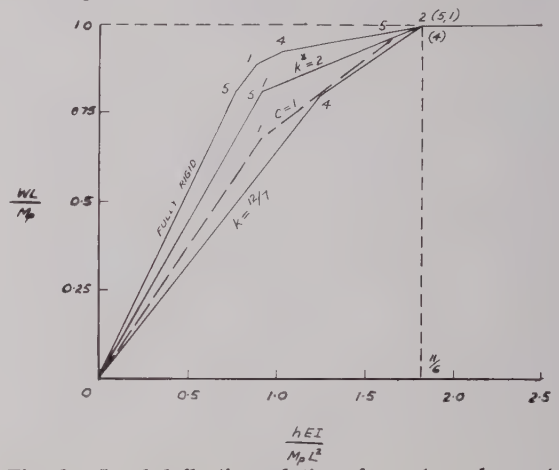


Fig. 9.—Load-deflection relations for rectangular portal frame.

For reference, the load-deflection relation for $k^* = 2$ is shown in Fig. 9.

Sinking of Supports

Consider now the situation which arises if each column base sinks under a vertical load V by an amount v, where $V = K\left(\frac{EI}{L^3}\right)v$, while rotation is completely prevented. From the nature of the loading it is evident that the vertical reaction at the right hand base exceeds that at the left hand base, so that the right hand base will sink relative to the left hand base. Fig. 10 shows the frame at the point of collapse, the relative sinking of the supports being d; it is assumed that the last hinge still forms at section 2. This implies that the distorted shape of the frame is precisely the same as in the fixed base case, so that in Fig. 10 the deflections relative to the rectangular datum 12′4′5 are the same as the deflections of the fixed base frame shown in Fig. 8. The rectangular datum 12′4′5 is obtained from the original position of the frame by a clockwise rotation $d/2L$, so as to lower the base 5 by a distance d relative to the base 1.

It will be seen from the figure that the effect of this rotation is to increase the plastic hinge rotations at both bases by an amount $d/2L$. Since d is essentially positive, there will be no critical value of d above which the signs of the plastic hinge rotations at the bases would conflict with the signs of the fully plastic moments, so that d, and therefore K, can have any value while leaving the deflections at the point of collapse relative to the datum 12′4′5 unchanged. In particular, the horizontal deflection of the beam relative to this datum would be $h_c = 11M_pL^2/6EI$, as shown, so that the total horizontal deflection would be $h_c + d$.

This conclusion may appear surprising, but it is borne out by the results of a step-by-step analysis. It is found that for any value of K, the sequence of plastic hinge formation is the same. Table 3 gives this sequence, together with the values of the non-dimensional load parameter WL/M_p and the non-dimensional deflection parameter hEI/M_pL^2 at each hinge formation, h being measured relative to the point 4′ in Fig. 10.

Table 3

Load-deflection relation for sinking of supports.

Section at which hinge forms	$\frac{WL}{M_p}$	$\frac{hEI}{M_pL^2}$
5	$\frac{84K+36}{103K+75}$	$\frac{80K-48}{103K+75}$
1	$\frac{140K+60}{157K+120}$	$\frac{424K-234}{471K+360}$
4	$\frac{40}{43}$	$\frac{136}{129}$
2	1	$\frac{11}{6}$

Spread of Feet

As a final example, the effect of spreading apart of the column bases will be considered. To make the problem specific it will be supposed that the left hand base remains absolutely rigid, whereas the right hand base moves horizontally by an amount HL^3/CEI, where H is the horizontal thrust, while the base of the column is still constrained to remain vertical. In this case a critical value of C exists below which the horizontal deflection at the point of collapse is increased, and this situation is depicted in Fig. 11.

If it is assumed that the horizontal deflection of the beam at the point of collapse remains unaltered, and that the curved shapes of the members are unchanged, the effect of a movement of the right hand column base is to rotate the right hand column 45 counterclockwise. This reduces the plastic hinge rotations at both sections 5 and 4 by the same amount. From Table 2 it is seen that the plastic hinge rotations at these sections in the fixed base case were respectively $7M_pL/12EI$ and $M_pL/2EI$ in magnitude, so that if the column 45 is rotated through an angle $M_pL/2EI$, as shown in the figure, the plastic hinge rotation at section 4 becomes zero while at section 5 a rotation of $M_pL/12EI$ in the correct sense remains. This, therefore, represents the greatest amount of rotation of the column which can occur in the manner described

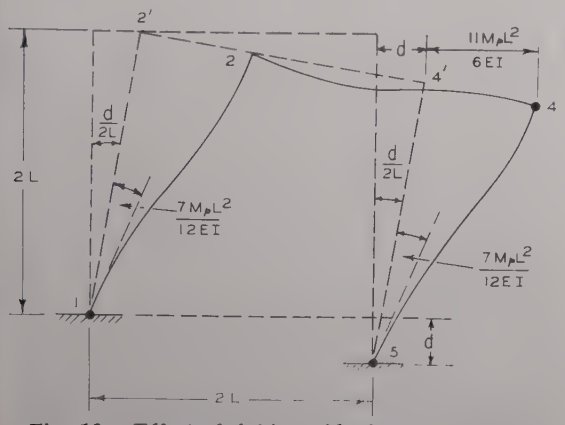


Fig. 10.—Effect of sinking of both column bases.

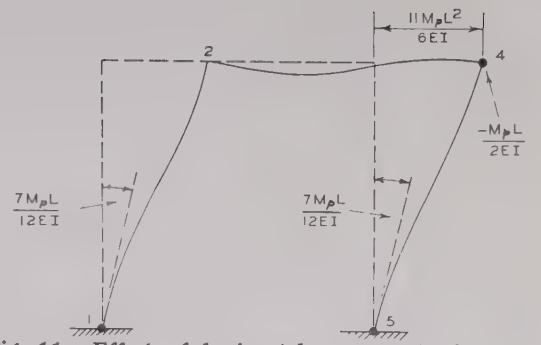


Fig. 11.—Effect of horizontal movement of column base 5.

without producing a plastic hinge rotation whose sense is contrary to that of the corresponding fully plastic moment. The corresponding lateral movement of the column base 5 is M_pL^2/EI, and at collapse the horizontal thrust at this base is M_p/L, so that if the horizontal deflection at the point of collapse is to remain unaltered,

$$\frac{M_pL^2}{CEI} < \frac{M_pL^2}{EI}$$

so that $C > 1$.

The load-deflection relation for the critical value of C, as obtained by a step-by-step calculation, is also shown in Fig. 9. It will be seen that at the critical value of C, hinges form at sections 2 and 4 simultaneously at the point of collapse. For smaller values of C, a plastic hinge will form at section 2 prior to collapse, and this would invalidate the above argument.

Conclusions

In designing steel frames by the plastic methods, it is sometimes of importance to determine the deflections which would be built up just before plastic collapse. If these deflections proved to be unacceptably large it would be illogical to design solely on the basis of the plastic collapse load, as the structure would reach a limit of acceptable deflection before the collapse load was attained. It is therefore significant that the deflections at the point of collapse may be unaffected by rotational flexibility effects at joints and column bases, even though such effects clearly reduce the stiffness in the elastic range and so cause increased deflections up to the point where yield first occurs. A simple method for estimating the rotational stiffness required of any given joint or base to keep the deflections at the point of collapse unchanged at the values for a fully rigid frame has been given. The technique involved has also been shown to be applicable to problems in which spreading or sinking of column bases may occur. The minimum requirements for combinations of stiffnesses has not been discussed, but the technique could easily be extended to apply to this type of problem.

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3. B. G. Neal, "The Plastic Methods of Structural Analysis." Chapman and Hall, London (1956), Ch. 8.

Discussion

The Council would be glad to consider the publication of correspondence in connection with the above paper. Communications on this subject intended for publication should be forwarded to reach the Institution by the 30th September, 1960.

Practical Design of Pile Groups

by A. Kroie, A.M.I.Struct.E.

Introduction

The problems connected with the design of pile groups are often the subject of lengthy and complicated analyses. This applies particularly to pile groups which have to withstand relatively large horizontal thrusts, where the use of raking piles would be advantageous. Because of the indeterminate nature of piled foundation structures, an exact analysis of structural members has not yet been possible ; and this presents a difficulty which is quite apart from the problems of soil mechanics involved. Investigations into the action of earth mass around piles and attempts to utilize the shear strength or the modulus of elasticity of soil for defining the ground resistance to horizontal thrust, do not appear to have yielded conclusive results of great practical value ; and easily applied design methods have certainly not yet been derived from them.

However, if the earthmass around piles is considered to provide no resistance to horizontal thrust at all, as might be true in the case of end-bearing piles in very bad ground, or if a known safe value of soil resistance is deducted from the applied horizontal thrust, and finer points of theoretical analysis are neglected, then the design of raking piles can be carried out quite easily and quickly.

The basic principles underlying the design methods here suggested are similar to those of graphical trial and error methods which are used for simple cases. In mathematical form, these principles can be applied to any arrangement of piles and a required pile rake can always be directly determined from the condition of loading.

The assumptions which must be made to facilitate design are—that pile caps are completely rigid and do not move or deflect from their original position, that piles are end-bearing and pinned at the pile cap ; and that analysis of any arrangement is possible by simple statics.

Pile Groups Carrying Vertical Load and Horizontal Thrust

The principle illustrated in Fig. 1 is that of a simple balancing of external forces by the vertical and horizontal components of the resultant pile loads. The rake at which the piles are driven is the only variable factor involved ; it is given by $-1 : x$, and its actual value may be determined from limiting conditions, that is maximum or minimum pile loads. Alternatively, a pile rake may be arbitrarily assumed and the resulting forces can then be quickly checked for safety.

It should be noted, however, that within limits, depending on the ratio of vertical load to horizontal thrust, the resultant pile loads will generally be least, if the piles are driven at the maximum possible rake. Hence, the greatest factor of safety against overturning will give the smallest resultant pile loads.

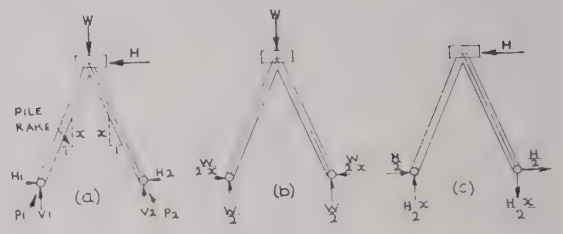


Fig. 1.

Referring to Fig. 1, and applying as limiting condition that no tension is to be allowed, then $-V_2 = 0$ and $H_2 = 0$.

Therefore $V_1 = W$ and $H_1 = H$

For equilibrium, rake required : $x = \dfrac{W}{H}$

and resultant pile load $= \sqrt{W^2 + H^2}$

e.g. if $W = 60$ tons and $H = 10$ tons,

then, rake required :— $x = \dfrac{60}{10} = 6$ (rake $= 1 : 6$)

and resultant pile load :— $P_1 = \sqrt{60^2 + 10^2}$
$= 60{\cdot}83$ tons, $P_2 = 0$.

Alternatively, if a maximum vertical component of 5 tons tension is to be permitted, then equating the vertical components of P_2 in Figs. I(b) and I(c)

$$\frac{W}{2} - \frac{H \cdot x}{2} = -5 \text{ tons.}$$

e.g. as before, $W = 60$ tons and $H = 10$ tons

then, $$\frac{60}{2} - \frac{10 \cdot x}{2} = -5$$

whence rake required :—

$$x = \frac{35}{5} = 7 \text{ (rake} = 1 : 7)$$



$$V_1 = 60 + 5 = 65 \text{ tons} \uparrow, \quad H_1 = \frac{65}{7} = 9{\cdot}3 \text{ tons} \rightarrow$$

$$V_2 = \frac{60}{2} - \frac{10 \times 7}{2} = 5 \text{ tons} \downarrow, \quad H_2 = \frac{5}{7} = 0{\cdot}7 \text{ tons} \rightarrow$$

Resultant pile loads :—

$$P_1 = \frac{\sqrt{50}}{7} \times 65 = 65{\cdot}65 \text{ tons compression,}$$

$$P_2 = \frac{\sqrt{50}}{7} \times -5 = -5{,}05 \text{ tons tension.}$$

If the maximum permissible pile load were limited to (say) 50 tons compression then the vertical components of P_1 in Figs. I(b) and I(c) would have to be equated thus :— $\frac{W}{2} + \frac{H \cdot x}{2} = +50$ tons

e.g. again, if $W = 60$ tons and $H = 10$ tons,

then, $\frac{60}{2} + \frac{10 \cdot x}{2} = 50$

whence, rake required :—

$x = \frac{20}{5} = 4$ (rake = 1 :4) $\sqrt{17}$ 4 / 1

$V_1 = \frac{60}{2} + \frac{10 \times 4}{2} = 50$ tons ↑,

$H_1 = \frac{50}{4} = 12 \cdot 5$ tons, →

$V_2 = 60 - 50 = 10$ tons ↑,

$H_2 = \frac{10}{4} = 2 \cdot 5$ tons ←

Resultant pile loads :—

$P_1 = \frac{\sqrt{17}}{4} \times 50 = \sqrt{17} \times 12 \cdot 5 = 51 \cdot 55$ tons

$P_2 = \frac{\sqrt{17}}{4} \times 10 + \sqrt{17} \times 2 \cdot 5 = 10 \cdot 31$ tons

The simple arrangement shown in Fig. I is given only to illustrate the method of determining the required pile rake, and because there can be little doubt as to the accuracy of the determined forces. For a more practical arrangement, that is, where the piles do not meet at a point in the pile cap, the applied horizontal thrust again may be equally shared between the horizontal components of raking piles. The results thereby obtained will be satisfactory for all practical purposes, even though this is somewhat difficult to prove.

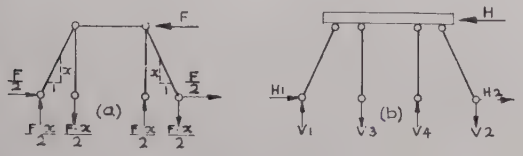


Fig. 2.

An analysis of an actual piling arrangement, such as shown in Fig. 2(b), by treating it as a redundant frame, will give contradictory results depending on which member is taken as redundant ; and the forces obtained, although providing perfect equilibrium, cannot be possible in practice. However, when the redundant frame in Fig. 2(a), is analysed by Castigliano's theorem, it will be found that the only variable forces involved are the vertical reactions, the value of which would depend on the actual rake of the inclined members. The horizontal components, on the other hand, are not essentially influenced by this rake and in all cases they are more or less of equal value. Therefore, if the assumption that pile caps are completely rigid is accepted as being correct, the conditions which influence the horizontal reactions must be identical in both cases. Although the vertical and raking piles in Fig. 2(b), do not meet at a point, the vertical piles, combined with the stiffness of the pile cap, provide the same resistance to movement as the vertical members in Fig. 2(a). Hence, all raking piles will resist the applied horizontal thrust with horizontal components of equal value.

The final forces in a pile group are determined by superposition as shown in the example.

Example I :—
Column Foundation.
Applied loads :—
$W = 400$ tons, $H = 15$ tons, $M = 0$.
12 pile group (Fig. 3), 6 piles raked as shown.

Pile loads due to W :—

$V = \frac{400}{12} = 33 \cdot 33$ tons

$H = \frac{33 \cdot 33}{x}$ tons

Pile loads due to H :—

$H = \frac{15}{6} = 2 \cdot 5$ tons

$V = 2 \cdot 5x$ tons

Again the pile rake can be determined by limiting conditions of pile loads. Thus, if the maximum permissible pile loads are (say) 50 tons, then—

$33 \cdot 33 + 2 \cdot 5x = 50$ tons

∴ Rake required :—

$x = \frac{16 \cdot 67}{2 \cdot 5} = 6 \cdot 67$ (use rake = 1 : 6) $\sqrt{37}$ 6 / 1

Maximum resultant pile load :—

$P_1 = \frac{\sqrt{37}}{6}(33 \cdot 33 + 2 \cdot 5 \times 6) = 49 \cdot 0$ tons.

In order to facilitate direct comparison of resultant loads with those obtained in example 2, assume a pile rake of 1 : 4. Then, final pile loads :—

$V_1 = 33 \cdot 33 + 2 \cdot 5 \times 4 = 43 \cdot 33$ tons ↑

$H_1 = \frac{43 \cdot 33}{4} = 10 \cdot 84$ tons →

$V_2 = 33 \cdot 33 - 2 \cdot 5 \times 4 = 23 \cdot 33$ tons ↑

$H_2 = \frac{23 \cdot 33}{4} = 5 \cdot 84$ tons ←

$P_1 = \frac{\sqrt{17}}{4} \times 43 \cdot 33 = \sqrt{17} \times 10,84 = 44 \cdot 67$ tons.

$P_2 = \frac{\sqrt{17}}{4} \times 23 \cdot 33 = \sqrt{17} \times 5,84 = 24 \cdot 06$ tons.

The final loads on intermediate piles may be determined from the above values by direct interpolation.

Pile Groups Carrying Vertical Load, Horizontal Thrust and Bending Moment

The additional bending moment acting on these pile groups does not at all add to the difficulties involved but, fortunately and perhaps unexpectedly, it provides a very useful medium for determining required pile rake.

If the horizontal thrust to which a pile group is subjected, is large enough to warrant the use of raking piles, then the greater part of the analysis follows familiar lines. That is, the maximum and minimum pile loads, due to vertical load and bending moment, are first determined by the well proven formula :—

$$P = \frac{W}{\text{No. of piles}} \pm \frac{M \cdot y}{I}$$

The pile loads thus determined are the vertical components of raking piles; they are constant and are not in any way influenced by the actual rake at which the piles may be driven. The ratio of horizontal components to vertical components, however, is entirely dependent on the angle at which the piles are raked. Hence, a greater pile rake will produce greater horizontal components and a greater unbalanced horizontal force in unequally loaded piles. The piles under consideration must, therefore, be driven at such a rake as to produce an unbalanced horizontal force which is equal and opposite to the applied horizontal thrust.

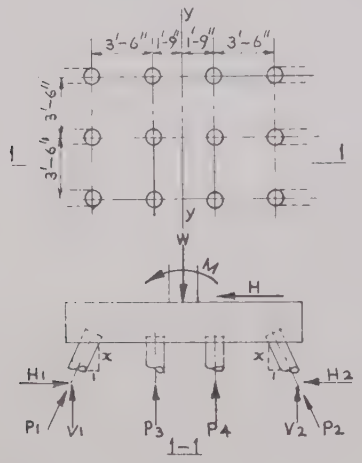


Fig. 3.

Again, by equating the horizontal components to the applied horizontal thrust, the required pile rake can be directly determined. Thus, referring to Fig. 3,—

$$\sum H_1 = \sum H_2 + H, \text{ and since } H_1 = \frac{V_1}{x} \text{ and } H_2 = \frac{V_2}{x}$$

$$\sum \frac{V_1}{x} = \sum \frac{V_2}{x} + H$$

whence, rake required :— $x = \dfrac{\Sigma\,(V1 - V2}{H}$

Example 2 :— Column Foundation.

Applied loads :— $W = 400$ tons, $H = 15$ tons, $M = 350$ ft. tons.

12 pile group (Fig. 3), 6 piles raked as shown.

$I_{yy} = 6 \times 1{\cdot}75^2 + 6 \times 5{\cdot}25^2 = 184$ ft.^{4}

Vertical pile loads :—

$$\begin{matrix} V_1 \\ V_2 \end{matrix} = \frac{400}{12} \pm \frac{350 \times 5{\cdot}25}{184} = 33{\cdot}33 \pm 10{\cdot}0 = \begin{matrix} 43{\cdot}33 \text{ tons} \\ 23{\cdot}33 \text{ tons} \end{matrix}$$

$$\begin{matrix} P_3 \\ P_4 \end{matrix} = \frac{400}{12} \pm \frac{350 \times 1{\cdot}75}{184} = 33{\cdot}33 \pm 3{\cdot}33 = \begin{matrix} 36{\cdot}66 \text{ tons} \\ 30{\cdot}0 \text{ tons} \end{matrix}$$

Rake required :—

$$x = \frac{3(43{\cdot}33 - 23{\cdot}33)}{15} = 4 \text{ (rake} = 1 : 4)$$

$\sqrt{17}$ / 4 / 1

Horizontal pile loads :—

$$H_1 = \frac{43{\cdot}33}{4} = \underset{\rightarrow}{10{\cdot}84} \text{ tons}, \quad H_2 = \frac{23{\cdot}33}{4} = \underset{\leftarrow}{5{\cdot}84} \text{ tons.}$$

The exact agreement of the above values with the forces obtained in Example I, and the perfect equilibrium between horizontal components and horizontal thrust, are due to the piles being driven at the exact rake required. When this is not the case, only approximate equilibrium can be obtained. For instance, a bending moment of 375 ft. tons, would cause a maximum variation in pile loads of :—

$$\pm \frac{375 \times 5{\cdot}25}{184} = \pm 10{\cdot}71 \text{ tons}$$

Therefore rake required :— $x = \dfrac{6 \times 10{\cdot}71}{15} = 4{\cdot}28$

In practice the piles would still be driven at a rake of 1 : 4, which would give horizontal components of slightly greater value than would be required to balance the horizontal thrust.

The next example outlines the variation of pile rake for different arrangements.

Example 3 :—Retaining Wall.

Data :—As shown in Fig. 4.

Net O.T.M./8 ft. run $= 36 \times 5 - 115 \times 0{\cdot}12 =$ 166·2 ft. kips.

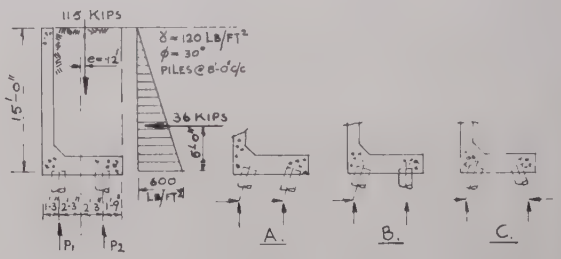


Fig. 4.

Vertical pile loads :—

$$\begin{matrix} P_1 \\ P_2 \end{matrix} = \frac{115}{2} \pm \frac{166{\cdot}2}{4{\cdot}5} = 57{\cdot}5 \pm 36{\cdot}9 = \begin{matrix} +\ 94{\cdot}4 \text{ kips.} \\ +\ 20{\cdot}6 \text{ kips.} \end{matrix}$$

Rake required: —

Case A :— $x = \dfrac{94{\cdot}4 + 20{\cdot}6}{36} = 3{\cdot}2$

Case B :— $x = \dfrac{94{\cdot}4}{36} = 2{\cdot}6$

Case C :— $x = \dfrac{94{\cdot}4 - 20{\cdot}6}{36} = 2{\cdot}05$

A known safe value of earth resistance deducted from the horizontal thrust would, of course, result in a smaller pile rake being required in each case.

For pile groups on which the bending moment and the horizontal thrust can act from any direction, it may be necessary to resolve further some horizontal components of raking piles. That is, those horizontal components which do not act in a direction perpendicular to the axis under consideration, must be resolved with respect to that axis. The parts of horizontal components which are active in resisting horizontal thrust are thereby determined and they have to be enumerated and expressed as a fraction of the piles. The required pile rake is then determined in the usual way. It should be noted that fully symmetrical arrangements will give similar results about any axis, as can be seen in the next example.

Example 4 :—Vessel Foundation.
Applied loads :—$W = 200$ tons, $H = 20$ tons,
$M = 600$ ft. tons.

8 pile group (Fig. 5), 4 corner piles rakes as shown, soil resistance to horizontal thrust = 0·5 tons per pile.
$I_{xx} = I_{yy} = I_{vv} + 6 \times 5^2 = 150$ ft.^{4}

Vertical pile loads :—

xx axis :—$P_1 = \dfrac{200}{8} \pm \dfrac{600 \times 5}{150} = 25 \pm 20 =$

$+45 \cdot 0$ tons, compression
$+\ 5 \cdot 0$ tons, compression

vv axis : $-P_2 = \dfrac{200}{8} \pm \dfrac{600 \times 5\sqrt{2}}{150} = 25 \pm 28 \cdot 28 =$

$+53 \cdot 28$ tons, compression
$-\ 3 \cdot 28$ tons, tension

Net horizontal thrust = $200 - 8 \times 0 \cdot 5 = 16$ tons.
Rake required :—

xx axis :—$x = 2 \cdot \dfrac{\sqrt{2}}{2} \dfrac{(45 - 5)}{16} = 3 \cdot 53$

(use rake = 1 : 3) $\sqrt{10}$ / 3 / 1

vv axis :—$x = \dfrac{53 \cdot 28 - (-3 \cdot 28)}{16} = 3 \cdot 53$

Resultant pile loads :—

$P_2 \text{max.} = \dfrac{\sqrt{10}}{3} \times 53 \cdot 28 = 56 \cdot 16$ tons, compression

$P_2 \text{min.} = \dfrac{\sqrt{10}}{3} \times -3 \cdot 28 = 3 \cdot 46$ tons, tension.

Check on horizontal components :—

vv axis :—Net $H = \dfrac{53 \cdot 28 - (-3 \cdot 28)}{3} = \dfrac{56 \cdot 56}{3}$

$= 18 \cdot 85$ tons.

It can be seen that for a pile rake of 1 : 3, the soil is only required to resist a total horizontal thrust of 1·15 tons. For piles driven at a rake of 1 : 4, the required soil resistance would be $20 - \dfrac{56 \cdot 56}{4} =$ 5·86 tons, or 0·73 tons per pile, which would be satisfactory for most practical cases.

Circular Pile Groups

Pile groups which support large chimneys or other tall structures, are usually symmetrically arranged on pitch circles. The analysis for these groups again follows the same principles as before. However, the work normally spent in determining the moment of inertia can be greatly reduced. It is not necessary to calculate actual pile distances from an axis in order to determine the moment of inertia, because it can be expressed in terms of the number of piles

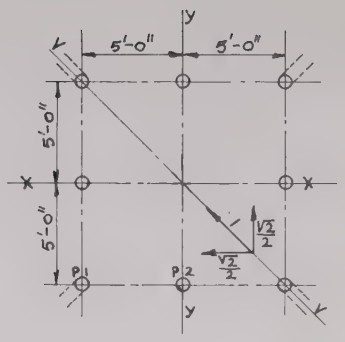


Fig. 5.

involved and the radius of the pitch circle on which the piles are spaced.

Namely $I = 0 \cdot 5 \times$ No. of piles $\times$ radius^2.

This is true for any number of piles, spaced on a pitch circle of any radius, and whether there is an odd or even number of piles does not matter.

e.g. Group of 13 piles equally spaced on a 16 ft. 0 in. dia, pitch circle,—

$I_0 = 6 \cdot 5 \times 8^2 = 416$ ft.^{4}

or $Z = 6 \cdot 5 \times 8 = 52$ ft.^{3} units.

While at first glance, the verity of the above formula may appear doubtful, a closer study of any symmetrical arrangement, such as shown in Fig. 5, which can be circumscribed by a circle or circles will show its accuracy for simple cases. Mathematical proof of the formula can be derived from circular sections in general. Thus, the moment of inertia of a circle, or of a circular ring, can be written as :—$I = \dfrac{A}{2} \times k^2$ where A is the area and k is the distance from the axis to the centre of mass. For a circle the location of this centre is where $-\pi (R^2 - k^2) = \pi k^2$.

Hence, $k^2 = \dfrac{R^2}{2}$, or $I = \dfrac{A \cdot R^2}{2 \quad 2} = \dfrac{\pi R^4}{4}$

Now, for piles (or columns, bolts or reinforcement) spaced on a pitch circle, the distance 'k' is, in fact, the radius. If the total area of pile cross sections were evenly distributed about the pitch circle, with the centre of mass being on that circle, the moment of inertia could then be written as :—$\dfrac{A}{2} \cdot R^2$.

Whether the total area is distributed over the circle or whether it is concentrated locally, cannot influence the results, provided that the radial centre of gravity remains unchanged. Therefore, $I = \dfrac{A}{2} \cdot R^2$ may be used for determining stresses in any member of circular structures where the centre of mass is coincident with the radius. For circular pile groups where, instead of stresses, direct loads are required, $\dfrac{A}{2}$ becomes $\dfrac{\text{No. of piles}}{2}$

Similar proofs could be derived from circular rings, for which the moment of inertia may be written :

$I = \frac{A}{2}(R^2 + r^2)$ and the formula may also be extended to hollow reinforced concrete sections where : $I_e = Ac(R^2 + r^2) + 7As \cdot R_1^2$. A full discussion on this, however, might be the subject of a separate paper.

Where raking piles are to be used, any axis through the centre of the pile group may be taken, and the horizontal components which do not act perpendicular to this axis must be further resolved, as has already been shown in Example 4.

Example 5 :—Chimney Foundation.

Applied loads : $W = 500$ tons, $H = 25$ tons,
$M = 1{,}500$ ft. tons.

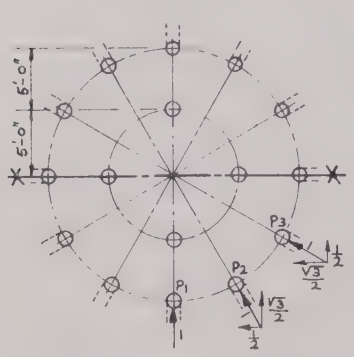


Fig. 6.

16 pile group (Fig. 6), all piles on outer pitch circle to be raked radially as shown.

$I = 6 \times 10^2 + 2 \times 5^2 = 650$ ft.[4]

Vertical pile loads :—

$$V_1 = \frac{500}{16} \pm \frac{1{,}500 \times 10}{650} = 31{\cdot}25 \pm 23{\cdot}08 =$$
$$+54{\cdot}33 \text{ tons, max.}$$
$$+\ 8{\cdot}17 \text{ tons, min.}$$

$$V_2 = \frac{500}{16} \pm \frac{1{,}500 \times 5 \cdot \sqrt{3}}{650} = 31{\cdot}25 \pm 20{\cdot}0 =$$
$$+51{\cdot}25 \text{ tons, max.}$$
$$+11{\cdot}25 \text{ tons, min.}$$

$$V_3 = \frac{500}{16} \pm \frac{1{,}500 \times 5}{650} = 31{\cdot}25 \pm 11{\cdot}54 =$$
$$+42{\cdot}79 \text{ tons, max.}$$
$$+19{\cdot}71 \text{ tons, min.}$$

Rake required :—

$$x = \Big[(54{\cdot}33 - 8{\cdot}17) + \frac{\sqrt{3}}{2} \times 2(51{\cdot}25 - 11{\cdot}25) + \frac{1}{2} \times 2(42{\cdot}79 - 19{\cdot}71)\Big] \times \frac{1}{25} =$$

$$\frac{46{\cdot}16 + 69{\cdot}28 + 23{\cdot}08}{25} = 5{\cdot}6 \text{ (rake=1 : 5)} \quad \sqrt{26}$$

Max. resultant pile load

$$= \frac{\sqrt{26}}{5} \times 54{\cdot}33 = 55{\cdot}42 \text{ tons.}$$

In practice, a safe working load of 60 tons would be specified for all piles.

Conclusion

There can be no doubt that further research, both in the field of soil mechanics and structural analysis, will lead to the development of more advanced design methods than have been discussed at present. However, although very formidable difficulties have to be overcome before design methods which may be applied to varying site conditions can be derived from soil mechanics, it is possible that model analyses of a number of piled foundation systems will yield informative results in the comparatively near future.

Discussion

The Council would be glad to consider the publication of correspondence in connection with the above paper. Communications on this subject intended for publication should be forwarded to reach the Institution by the 30th September, 1960.

Book Review

The Theory of Thin Shells, by Professor V. V. Novozhilov. (Groningen, The Netherlands : P. Noordhoff Ltd.). 376 pp., 36 guilders, approx. 75s.

Translated from the Russian by P. G. Lowe formerly of the University of Sydney, and edited by Professor J. R. M. Radok of the Polytechnic Institute of Brooklyn, this is the first book in English in the reviewer's knowledge to give a general theory of thin shells. Written almost ten years ago and published in Russia in 1951, it will be read by all engaged in shell design with very great interest. (A new Russian edition has recently been published which is understood to contain additional material).

In the four sections of his book, Professor Novozhilov deals with the general theory of thin shells, the membrane theory, the analysis of cylindrical shells, and the analysis of shells of revolution. In the description of the historical development of shell theory, references are made to such well known authors as Love, Dischinger, Finsterwalder and Schorer, but these are outnumbered several times by the references to Russian writers in this field. The amount of work carried out in Russia on this subject may be gauged from the fact that two other books have been published in Russia on shell theory, one by V. Z. Vlasov and the other by A. L. Goldenweizer.

The translation, editing and presentation of this authoritative work by Professor Novozhilov is of a very high standard and those concerned are to be congratulated. It is hoped that the success of this venture may encourage the translation and publication of many other Russian books of scientific and engineering interest. C.R.

The Structural Engineer in the Field of Atomic Energy *

Discussion on the Paper by T. C. Waters, M.I.Struct.E. (Delegate Member of Council)

Discussion

THE PRESIDENT said the meeting would agree that they had listened to a very able introduction of a most informative paper. A good deal of work in connection with this subject had passed and was passing through Mr. Waters' hands and it was being dealt with in the efficient and quiet way which was symbolised by the efficient and quiet way in which he had introduced the paper.

This was a field which was comparatively new and very British and engineers must all be very proud of that.

He invited the meeting to show, as indeed they should, their appreciation of the great trouble which Mr. Waters had taken to amass so much information in the paper, for his presentation of it and for the very good pictures he had shown.

Mr. K. H. BRITTAIN (Associate-Member), commenting on the reference on page 37, offered any information required on the heat of hydration of cements supplied by his Company.

Pointing to the reference, on page 35, to the scatter of fast neutrons by the hydrogen nuclei present in the water of hydration, he asked if the entrainment of hydrogen in heavy concrete by adding aluminium powder to the wet mix had been considered.

On page 38 there was reference to the question of the permeability of concrete to gases and liquids. Mr. Waters was concerned with the leakage of low pressure air or gases; many others were concerned with the infiltration of water vapour or water into basements, the penetration of chemically aggressive liquids into concrete, and the action of frost on concrete which had a high absorbed water content. If Mr. Waters could make leak-tight concrete many people would be interested and Mr. Brittain asked if the UKAEA would make further information available in due course.

Mr. WATERS said he was aware that a lot of work had been done on the subject of heat of hydration, we knew that information existed and we knew its sources. His point was to ask if we really took it into account in assessing the stresses in buildings which were subjected not only to structural loadings but also to temperature gradients. He had purposely mentioned the high heat of hydration recorded at Calder Hall because the figures had surprised him and he had concluded that although we knew the problem existed this was a subject with which we never seemed to get to grips.

Replying to the question concerning the presence of hydrogen for dealing with fast neutron scatter, he said that ten years ago the nuclear physicist was very concerned that hydrogen should exist in sufficient quantity. We now knew that provided it existed in the proportion of at least 0·5 per cent of the volume of the shield—as indeed it did in any normal concrete—the concrete shield was perfectly capable of performing the job for which it was needed, therefore concrete automatically became a suitable material for this aspect of shielding.

The problem of permeability or leak tightness, was one which had occupied his mind for a long time, and he had engaged in some *ad hoc* experimental work on this subject to the extent that he had endeavoured to arrive at some kind of relationship between the pressure of enclosed gas and the thickness of the barrier through which it had passed. From these tests he had arrived at a straight line formula something like this :—

$$L \text{ (leakage rate)} = 12 \cdot 5 \frac{p}{t} \text{ cu. ins./sq. ft./hour, where}$$

L = leak rate in cubic inches per sq. ft. per hour

p = pressure (p.s.i.g.)

t = slab thickness in inches.

Whilst this relationship was useful it should be treated with reserve for the more one considered this problem and the bigger the range of specimens used in the tests, the wider was the scatter in the results obtained. He was prepared to make the point that the smaller the test specimens used the better the results on impermeability.

However, it was sufficient to say that with this formula he had calculated that about 1 per cent leakage through the HERO container could be anticipated in 24 hours. He said that with his tongue in his cheek because the interpretation of these test results had to be related to the problems on site of the supervision of concrete mixing, placing, vibration, and curing; each and every one of these operations contained somewhere that element of possible defect which could slip through unnoticed but which could materially affect the result.

Turning for a moment from concrete to steel plate, he said that in spite of the very tight control exercised in its analysis, ultrasonic testing against concealed defects, independent inspection of welders and welds with all that that implied, it was just about possible to give the reactor designer the leak-tight integrity that he asked for, namely 0·1 per cent in 24 hours, and this occurred through door seals, penetrations, etc.

Coming back to concrete again, he felt that it could not compete with steel on equal terms as a medium for containment, and we had really got to start again. He was prepared to open up on that later if necessary.

Mr. L. C. WARD (Associate-Member) asked what particular precautions were taken on the inside of a concrete containment vessel to prevent contamination

* *Read before the Institution of Structural Engineers at* 11, *Upper Belgrave Street, London, S.W.*1, *on the* 28*th January*, 1960. *Mr. L. E. Kent, B.Sc., M.I.Struct.E., M.I.C.E. (President) in the chair. Published in "The Structural Engineer" Vol.* XXXVIII, *No.* 1, *pp.* 26-40 (*January*, 1960).

by CO_2 or other coolant gas. With other pressure vessels it was a fairly simple process; but he asked whether in this case a particular method was used to remove contaminants from the coolant gas.

Mr. WATERS replied that the coolant was contained within the pressure vessel, heat exchangers, and connecting ducts forming the primary circuit. We were alive to the need for keeping the inner face of the concrete biological shield clean, and the ventilation system which was used to cool this face contained filters to clean the air before it was passed to atmosphere. It would be appreciated that the present system of pressurised reactors involved an enclosed circuit for CO_2, and no leakage would be expected to occur, relatively speaking. That was one of the reasons why such stringent conditions were laid down both for the design of the primary circuit and for its fabrication.

Mr. P. A. BADLAND (Member) asked what kind of concrete the author had in mind for use for containment buildings, whether any special specifications had been suggested and, in particular, whether the use of any special additives had been considered.

Mr. WATERS said they had to ensure the production of a concrete having a consistency which would allow it to be placed and compacted to give a specified density. To that extent the choice of each site governed the conditions there, both with regard to the materials reasonably available and the plant that would be used. On HERO at Windscale they had adopted a mix which was designed to give a specific density of 145 lbs, per cubic ft. with due regard to impermeability, and they were using a mix of 8 : 1. In designing this mix they had found that as between 25 per cent of sand to the aggregate as a whole, and 34 per cent of sand to the aggregate as a whole, the increased sand content gave a better impermeability factor when this particular mix was adopted.

Additives had been used for a long time; they were still being used and would continue to be used. He had a feeling about additives—which might be wrong—that they might make it easier to produce good concrete, and that they might improve bad concrete, but frankly he did not believe that they would improve otherwise good concrete. He was not at all clear that, on the long term basis, whatever additive was used one would necessarily get value for money, and in considering containment problems we must think in terms of the long term aspect.

He did not know whether Mr. Badland had in mind normal reinforced or prestressed concrete, but considering normal concrete to begin with his feelings were that if we were contemplating the use of concrete for containment purposes we could consider in the first place the conditions likely to arise when using steel plate, and then to consider them in relation to the conditions which were likely to develop if steel plate were replaced by concrete.

For example, the Dounreay Sphere which was in steel plate varying from 1 inch to $1\frac{3}{4}$ inch thick, was based on a design which postulated that under accident conditions the air temperature in the sphere would rise to 300°C, and that at this temperature an internal pressure of 18 lb/sq.in. could be expected. What set of design conditions would arise if we used concrete? To begin with it would be rather difficult to design a concrete building economically to cater for this pressure condition alone, but if we finished with a 12 inch wall thickness the heat transfer characteristics of this wall would be so poor by comparison with that of the steel membrane, that the 300°C air temperature design condition would be far exceeded, and as a consequence the pressure design condition would be far exceeded, In other words, we have changed the conditions of design. We should therefore have to refer back to the reactor designer and inform him that the temperature conditions could not be tolerated. Again Mr. Waters said that an internal air temperature above 100°C would not be acceptable to him for a concrete containment building.

If we were to seek reductions in the incident temperature in order to reduce the resultant pressure we should find ourselves either with a bigger building to provide a big increase in contained volume, or we should have to ask the reactor designer to start re-thinking on his philosophy of containment. It was possible for him to do this of course, but up to the present time he had preferred a solution which was clearly defined both in structural and in containment terms. The sort of questions we should have to put to him were whether he would be prepared to provide some system of depressurisation within the containment building, which would come into operation automatically in an emergency, and whether he could conduct the fission products through scrubbers and then to allow the clean outlet gases to be blown to atmosphere. This would enable the structural engineer to provide a concrete containment building which would have a less onerous duty to perform by comparison with the steel one, but which might well serve the purpose of containment. It tended to become rather involved because we were asking the reactor designer to provide additional expensive plant and equipment. He would be concerned about the reliability of this equipment and the capital and maintenance costs of equipment, and housings, would need to be weighed against the savings in the concrete building. He might feel that we had passed the ball back to him.

With regard to the use of prestressed concrete, Mr. Waters thought that it would improve the impermeability of the concrete membrane. He did not think, however, that we could do very much to cater for higher temperature gradients.

Referring again to the Dounreay Sphere, he said that when considering the use of concrete for containment, we obviously would not think in terms of a sphere but rather of a cylinder. When we considered a cylinder we had to think about the anchorage problems and we were back where we had started, for it had been found at the outset that the anchorage of large cylindrical containment buildings presented very formidable design problems.

Mr. K. S. SOMMERFELD (Member) wondered whether Mr. Waters had considered a system which, one understood, the French had introduced some 20 years ago, consisting, roughly, of an ordinary hollow wall construction; in other words, there were two walls of concrete with a gap between them, filled with water. Such a scheme might help to solve some of the problems which the U.K. Atomic Energy Authority were interested in. He imagined that liquids other than water could be used in the cavity, and furthermore, whatever liquid was used could be circulated to dissipate the heat that Mr. Waters had mentioned.

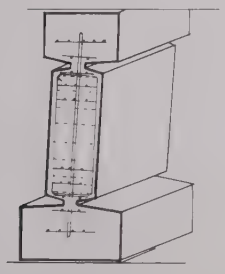
MR. WATERS said he did not know of the French system to which Mr. Sommerfeld had referred, and up to the present time the U.K.A.E.A. had not considered using a double skin for containment purposes but they had given quite a bit of thought to a double skin for biological shielding. To some extent he supposed anything was possible but with any given problem one sometimes met different aspects of design which would rule out what would otherwise be a likely solution.

There were certain virtues in a liminar construction. By providing an inner unrestrained membrane we might perhaps more easily cater for the temperature deformations (the steel thermal shield at Calder acted in this capacity) and we might provide a cavity for ventilation as a means for disposing of heat. But in regard to the shield, we had to provide a barrier against radiations from the reactor itself and a minimum of thermal movements, as there was a necessity to ensure that we were not creating undue stresses within the ducting and other penetrations through the shield, and thus throwing the problem back to the reactor designer.

MR. D. H. NEW (Member) recalled a reference to a Freyssinet hinge and asked if Mr. Waters could illustrate the particular application.

In connection with the Reactor known as HERO the author had mentioned the use of water bars. It would be interesting to know whether in fact, there was any appreciable radiation and if so, what material was used for these bars ; were there objections to the use of rubber under certain circumstances ?

MR. WATERS illustrated the Freyssinet hinge. It was cast, he said, in very high grade concrete and the stressing across the neck was accepted up to yield. He showed the rotational effect.



The Freyssinet hinge showing mode of rotation

With regard to water bars and the effect of radiation on rubber or P.V.C., he said that some special P.V.C. water bars had been used. They were produced by the usual manufacturers who were given a specification laid down by the nuclear physicists ; the chemical formula was adjusted to satisfy them in respect of radiation effect. The type used in the Windscale Pond exhibited certain disadvantages, particularly with regard to a tendency to brittleness adjacent to vulcanised joints.

There would appear to be no objection to the use of rubber or P.V.C.; but it was important to get this checked by the reactor designer with whom one was working in case there were likely to be radiation conditions which might be critical.

PROFESSOR P. B. MORICE (Associate-Member), referring to the work on the stability of spherical shells, said that all shells seemed to have a critical load well below what was predicted from simple theory. This was, no doubt, due to the deviation from sphericity in practical structures. A graph exhibited by Mr. Waters appeared to show, amongst other curves, one which gave a reasonable prediction of practical critical stress values. Would Mr. Waters say what was the basis of this particular analysis. It would be interesting if he could explain a little further what he had in mind when he had made a plea for further research into this subject. Professor Morice believed that the Engineering Department at Cambridge was doing some work on this subject, one of the problems, he believed, being the production of a test specimen with uniform thickness and reasonable sphericity.

MR. WATERS replied that one of the curves he had shown related to some experimental work carried out by Von Karman on small copper spheres, and this was probably the curve to which Professor Morice referred. Another represented what was called the Hortonsphere formula based on analogy to column tests. A further curve was produced from the results of tests to failure of a 52 ft. 6 in. diameter commercially built Hortonsphere. With regard to this latter the lack of sphericity in one area was greater than that specified by the designers, and it was because of this that the makers had allowed themselves the luxury of testing to failure.

As to the research on the sphericity of shells, Dr. Chilver at Cambridge had a research programme in hand at the present time on the stability of spherical shells, and had developed a cunning device for producing very small accurate spherical shapes of uniform thickness. Mr. Waters had persuaded Dr. Chilver to undertake a series of experiments which it was hoped would lead to a better understanding of the effects of deviation from true sphericity upon stability against buckling. Vacuum tests will be carried out upon large dome sections manufactured as accurately as possible with known deformations. He imagined that something like two years might elapse before we were able to benefit from any information on this problem.

He felt very strongly about the question of deviation from true sphericity where buckling was a criterion. It assumed a major importance on a sphere, say, $\frac{1}{2}$ in. thick and 135 ft. in diameter, as in the case of the A.G.R. One could imagine the manner in which sections would deform like wafers when they were lifted. It was quite a problem to cold press them into shape in the shops because of backspring, and he believed the manufacturers had learned quite a lot from recent experiments on the subject of cold pressings of this magnitude. The structural engineer had to make an assessment at design stage of the degree of deviation he would permit and then to convince himself that he had made due allowance but not more allowance than was necessary, bearing in mind all that went into the fabrication and erection of the building. He had to consider all the past history of the completed sphere, in fabrication the effort to get the plates to the correct shape in the first place, and their handling in the shops by cranes ; there might also be a certain amount of additional fabrication on site which would include welding, the assemblies would have to be lifted and keyed into position and aligned. Weld procedures had to be produced to bring the welded assemblies truly into shape, avoiding unacceptable peaking or flattening between welded plates. These aspects had to be considered before design in arriving at acceptable shape tolerances. If one were too generous there would be too much deviation from true sphericity with a resultant uneconomic design leading to unnecessary thickening of plate ; on the other hand, one might ask a contractor to produce something which was literally impossible on a commercial basis and to waste money attempting to do it.

PROFESSOR MORICE commented that the critical stress could perhaps be increased most readily by using ribs. He asked if there were any special reason why circumferential ribs were not provided.

MR. WATERS replied that possibly Professor Morice's remark was true. In the case of the "pear drop" containment building, ribs were provided in the truncated cone supporting the hemisphere.

In regard to the spherical shape, he and his colleagues had endeavoured as far as possible to avoid accumulated bending stresses which occurred when ribs were used. Maybe this approach was wrong, but on paper they were satisfied that it was the more economical.

MR. J. R. DUFFELL (Graduate) drew attention to the importance of the foundations. Mr. Waters had referred to reactor buildings involving loads of 7,000 tons and upwards. How was all that load dispersed in the foundations?

Also he invited Mr. Waters to expound on some of the points he had made in the paper about settlement, for there were fairly excessive loads which could bring about appreciable settlement in the ground. One realised, of course, that this was a very general subject; but it was a very important aspect, for example in respect of the biological shield, where cracking had to be avoided at all costs for safe working and operation.

MR. WATERS said he believed that what was regarded as good practice in respect of foundation design for any normal building had been applied to the buildings discussed in the paper, particularly those which tended to be rather massive, such as nuclear reactor buildings. Those concerned in the design took a great deal of care to ascertain the characteristics of the material they were working in. Soil mechanics reports were prepared on the subsoils, long term settlement problems were considered, and due allowance made for that aspect by restricting the imposed unit load.

With regard to the massive project at Harwell in connection with which he had referred to differential settlement, he said that considerable care was devoted to the site investigations. Rather fortunately it was in chalk rock but, even so, considerable long term settlement could be expected on that site. But the important thing from the structural engineer's point of view was to collect his important and co-related items on to one foundation, thereby avoiding differential settlement between them. His prime problem then was to ensure that he did not get tilting.

In the case of A.G.R. all the important loads were brought down on to one foundation. At Windscale designers had become familiar through the years with the ground conditions, and they worked to something like 2 tons per sq. ft. mean pressures rising to edge pressures of $2\frac{1}{2}$ tons per sq. ft. in certain circumstances.

So that the general answer to Mr. Duffell was that they tried to apply what was commonly regarded as good engineering practice.

MR. O. T. BARFORD (Member), referring to the concretes used for biological shielding, of which particulars were given in Table 1, asked for Mr. Waters' views on the use of magnetite as an aggregate. It was frequently referred to in American literature he said, and on the Continent it had been used recently.

MR. WATERS said that he knew that a good deal of magnetite had been used for heavy shielding on the other side of the Atlantic and on the Continent. His organisation had not used much of it, primarily for the reason that if they needed heavy aggregates they tended to go to sources readily available in the United Kingdom. They had used barytes which had given fairly good concrete and was easily workable, although in storage it was rather friable. The market was not particularly good because its production was on a very small level when compared with that of normal aggregates.

He believed that magnetite was available from Scandinavia but was not quite sure. Provided the quantities were favourable at the right time and that the price was all right, he would be glad to use it. Ilmenite was another heavy aggregate which might have certain advantages over magnetite and it did not involve the problem of friability; furthermore, on market prices today, it was probably more economic.

THE PRESIDENT, closing the meeting, said it had been an interesting discussion and the replies to the questions had been exactly what the meeting would have wished, delivered by Mr. Waters in his very efficient way. They would all wish to thank him for his replies.

Corrigendum

The Structural Engineer January 1960, p. 38, col. 2, line 19 under the heading "Concrete Containment Buildings." *For* 5 per cent of the volume within 24 hours *read* 2 per cent of the volume within 24 hours.

Book Review

Surveying, by A. Bannister and S. Raymond. (London, Pitman, 1959). $8\frac{3}{4}$ in $\times$ $5\frac{3}{4}$ in., 438 pp. 45s.

The primary object of Messrs. Bannister and Raymond in writing this text book is to meet the needs of the student who is reading for professional examinations. It is pointed out in the preface that 'Surveying' will form a companion volume to 'Advanced Surveying,' the latter by A. Jameson.

'Surveying' shows how well the authors have succeeded in producing a modern text book which is both concise and full of interest. A very wide range of subjects is covered, chain surveying, levelling, the theodolite and transverse surveying, surveying instruments, tacheometry, curve ranging, triangulation surveys, hydrographic surveying, photogrammetry, field astronomy and the theory of errors. The many drawings and photographs throughout the book are well positioned in the text, which makes for easy reference.

After the explanation on each subject there follows a set of exercises and answers which give the student the opportunity of proving his own absorption of the reading matter, providing he is determined enough in himself to cover the answers.

The usefulness of the book also extends beyond the student to the Engineer and Surveyor in practice who desires to keep up to date with new methods, new instruments and the advancement in knowledge made by the optical scientist, the instrument maker and others.

This text book will prove to be a very good addition to the office or private library, and the authors are to be complemented on the result of their labours, also the publishers and printers on their setting out and production of a clear and readable edition.

H.H.B.S.

A Convergence Technique for Determining the Elastic Critical Load of Rigidly Jointed Trusses*

Written Discussion on the Paper by Arthur Bolton, M.Sc.Tech., Ph.D. (Graduate)

MR. M. GREGORY commented that the slow convergence or divergence of the moment distribution process at loads near the critical load had tended to make the calculation of critical loads for elastic rigidly jointed frames by this method rather tedious. He quoted the reason Dr. Bolton had given : " Basically this difficulty arises because the testing distortion used is not the critical mode, but merely the rotation of one joint. If the critical mode were to be used as the testing distortion, one cycle would be sufficient to decide whether the calculations were converging or not." Mr. Gregory thought this had become clear to most people who had used the Hoff moment distribution convergence criterion. It took many distributions for the effect of a single disturbance to be felt throughout the whole structure, and many more for the carry-over to reflect back to the originally disturbed joint, and the first step towards reducing the length of the calculation was obviously to apply disturbing moments at several joints rather than at one joint, particularly if the disturbances could be given the correct sign, which was often the case if the desired buckling mode could be pictured. This method had previously been used in the University of Tasmania, but it was interesting to see it carefully worked out in detail in Dr. Bolton's paper, particularly with regard to the information which could be gained by keeping running totals of the disturbing moments after each distribution, and he complimented Dr. Bolton on this advance.

Mr. Gregory emphasized that the calculation of critical loads (' the load at which deformations would increase without limit even if there were no initial imperfections and the structural material were infinitely strong ' is the definition given in the paper) was only a means to an end. In practice one was usually interested in the load-carrying capacity of structures possessing practical imperfections and exhibiting yield, rather than the mathematically defined critical load of the perfect structure. He hoped that it would eventually be possible to define the failure load of a structure in terms of its critical load and its practical imperfections. Various suggestions had been made. (References [1] to [6]). The critical load thus retained considerable importance. It should be remembered however, that the initial crookednesses of members and the eccentricity of loading of a structure could cause it to fail by buckling in a mode which was not the mode possessing the lowest critical load. This was particularly evident in problems of instability of framed structures in which buckling of members in flexure and torsion was involved. A plane triangular frame for example could buckle out of its plane in at least two modes the critical loads for which could differ by fifty per cent, and certain initial crookedness patterns might cause the frame to fail with deformations assuming the mode which had the higher critical load. Such higher modes should not always be regarded as mathematical fictions, as they could present very real solutions to practical problems without the necessity of introducing artificial restraints.

He referred to the following sentence which occurred on page 233 of the paper ; " This paper describes a method of determining critical loads in the elastic range which does not concern itself with the stiffness of the frame but with the convergence or otherwise of the calculations." The inference from later paragraphs was that the method given had more physical meaning than some other methods. Mr. Gregory wondered whether the common mathematical foundation of all these methods of calculating critical loads was generally realized.

Considering the eigenvalue problem given by the n linear homogeneous equations in $x_1, x_2, \ldots x_n$, where the coefficients a_1, b_1, etc., were prescribed functions of some variable A.

$$\left.\begin{aligned} a_1 x_1 + a_2 x_2 + \ldots\ldots + a_n x_n &= 0 \\ b_1 x_1 + b_2 x_2 + \ldots\ldots &= 0 \\ \vdots \\ n_1 x_1 + \ldots\ldots &= 0 \end{aligned}\right\} \quad \cdot \quad (1)$$

Then, in general, the variables x could take only zero values. But for definite (and usually required) values of A, there existed non-zero solutions. This occurred

when $$\begin{vmatrix} a_1\, a_2 \ldots\ldots a_n \\ b_1 \\ \vdots \\ n_1 \end{vmatrix} = 0 \quad \cdot \quad \cdot \quad (2)$$

This condition might hold for various values of A such as A_1, A_2, etc., if either the determinant was factorizable or the parameters a, b, etc., were periodic. In any case, when (2) held, the n equations were not independent, and hence the variables x were undefined.

In the case of a framed structure containing members such as AB, we had equations relating the end slopes θ_{AB}, etc., and the end moments M_{AB}, etc., such as

$$AB = \frac{M_{AB} l}{3EI}\,\beta - \frac{M_{AB} l}{6EI}\,\alpha \quad \cdot \quad \cdot \quad \cdot \quad (3)$$

where α and β were functions of the axial load in AB. (Extra terms must be included if there were twisting moments). Writing down all such equations for all members, and equating the rotations of the members at the rigid joints, a system of equations similar to (1) was obtained such as

$$\left.\begin{aligned} a_1 M_1 + a_2 M_2 + \ldots\ldots &= 0 \\ b_1 M_1 + b_2 M_2 + \ldots\ldots &= 0 \\ c_1 M_1 + c_2 M_2 + \ldots\ldots &= 0 \end{aligned}\right\} \quad \cdot \quad \cdot \quad (4)$$

* *Published in " The Structural Engineer " Vol. XXXVII, No. 8, August 1959.*

where the parameters a_1, b_1, ..., were functions of the loading on the frame. That is A defined the load on the frame. When the determinant (2) was zero, the variables M were undefined; hence the linearly related slopes θ were undefined. The frame had buckled, and using the classical method (2), could be used to obtain the critical loads.

All 'convergence' or 'stiffness' methods were directly based on the properties of linear equations similar to (3) or (4). If we put

$$a_1 M_1 + a_2 M_2 + \ldots + a_n M_n = M_0$$

keeping the other equations in (4) unaltered, then the condition that $\frac{M_1}{M_0}$ or $\frac{M_2}{M_0}$ became infinite, was the same as equation (2), and occurred at the same values of the loading A. That was the moment distribution convergence criterion. M_0 was a disturbing moment. For $A < A_1$, $\frac{M}{M_0}$ was finite. For $A \leq A_1$, $\frac{M}{M_0}$ became infinite. Moment distribution was merely an iterative method of solving equation (2).

Dr. Bolton's method corresponded to applying disturbing moments at more than one joint, or putting

$$\left.\begin{aligned} a_1 M_1 + a_2 M_2 + \ldots\ldots &= M_{01} \\ b_1 M_1 + b_2 M_2 + \ldots\ldots &= M_{02} \\ c_1 M_1 + \ldots\ldots &= M_{03} \end{aligned}\right\} \text{ etc.} \quad . \quad (5)$$

The critical loads were still given by $A = A_1, A_2, \ldots$.

Allen's method consisted of dealing with slopes and moments, or equations of the type of (3), throughout. In the end two linear equations were obtained and neutral equilibrium existed when zero energy was required to rotate a joint, that is $\frac{1}{2}M_A\ \theta_A$ is zero (Reference (7)), or the stiffness of the joint $\frac{M_A}{\theta_A}$ is zero. (Reference (8)).

Allen had shown that the method was mathematically the same as reducing a determinant.

The methods of obtaining the critical load were thus mathematically the same, but it was most interesting to read papers such as Dr. Bolton's where by the application of a suitable technique the work of calculation could be considerably lessened.

Dr. Bolton replying thanked Mr. Gregory for his discussion, which he felt would clarify several points in the original paper and at the same time fill in some of the background which was omitted. Referring to Mr. Gregory's statement that the calculation of critical loads is only the means to an end, Dr. Bolton commented that it was difficult to write a research paper which was of immediate use to more than a small minority of engineers. All the same, these papers must be written and discussed, particularly when they were part of a larger programme which was casting doubt on current assumptions in structural design. That was the present situation in respect of critical loads. It could be clearly shown that in many designs a considerable expenditure was wasted on stanchions which were relatively stronger than the rest of the structure. Moreover, the same codes which wasted this material could produce other structures which were entirely unsafe. Dr. Bolton suggested that any engineer in doubt of this fact should inspect photographs of the kind shown in *Engineering News Record* of 11th September, 1958 and 14th May, 1959, if only for reasons of self-interest.

In the former case an extensive eleven-storey portal with welded joints collapsed in a series of planes like a pack of cards when the bare frame was almost complete. There was hardly any wind at the time, and even after such catastrophic failure only two of the welded joints were found to be broken. Yet the weight of the frame itself was sufficient to cause lateral collapse. It was easy to say that the frame should have been more securely braced with wire ties until the cladding was complete, and any prudent engineer would agree, particularly after the crash. But this did no more than obscure the important underlying problem. The structural framework, about which all the calculations of strength and stiffness were performed, was unable to carry its own weight without the support of the cladding. He wondered then how close to failure the whole building was when complete, and whether the supporting cladding needed to be brickwork panels, lightweight concrete blocks, curtain walling or $\frac{1}{4}$ inch thick plate glass.

He thought perhaps the traditional education of structural engineers was at fault. They had all seen struts tested in testing machines and were used to the idea that critical loads were about the size of the Euler load. Effective lengths of stanchions prescribed by design codes gave the same idea. But it was now technically possible and architecturally desirable to build frames in which the critical load was only a fifth or a tenth of the Euler load, and they were being built both in steel and concrete. In fact, it was rather doubtful which would come first, rational stability design or the catastrophic failure of a new building.

If it must be the latter it would be catastrophic in two ways. Firstly, a critical load collapse was sudden and complete. To observe such a failure in the laboratory with enough imagination to translate the effects to a full size building could be frightening. Secondly, such a failure, in a building which complied in every respect with the appropriate design code, would disrupt the orderly advance of structural engineering in the same way that Tacoma Narrows had hindered the building of suspension bridges even when they had been proved aerodynamically safe. Dr. Bolton suggested it would be interesting to hear the views of members of the various design code committees on this topic.

The urge to remedy the present state of affairs started in Manchester. More than ten years earlier Professor Merchant put forward, in undergraduate courses at the College of Technology, what he has called the Generalised Rankine formula. Dr. Bolton, on graduation, had been fortunate enough to join him from the beginning of this relevant and fruitful research and to see many other people drawn in. From the first it was recognised that the greatest obstacle to rational design was the lack of simple methods of calculating critical loads. The paper under discussion was merely the latest published of a long series aimed at improving elastic and critical load analysis. The Department of Building and Structural Engineering was fortunate because there was in the Manchester Computer Laboratory at that time Dr. R. K. Livesley, who produced, with Dr. Chandler, the excellent tables of functions for axially loaded members. A serious experimental programme was started only after the calculations had been shown to be sufficiently practical for design use and it had been carried out in a particularly elegant series of calculations and experiments by Dr. A. Salem in the Department of Structural Engineering at Manchester, by Dr. Bolton himself at Liverpool University over the past five years, by Mr. C. A. Rashid in the Department of Structural Engineering, and Dr. Murray and Mr. Nutt in the Department of Civil Engineering at Manchester. Lately, several workers at Cambridge University and elsewhere had also carried out significant work

confirming the Rankine formula, sometimes, it seemed, to their own mild astonishment.

With regard to initial eccentricities and possible failure in a higher mode, Mr. Gregory was perfectly right as far as laboratory tests were concerned. To link an experimental failure load with calculated critical and plastic collapse loads it was necessary to consider modes other than the lowest if these occurred due to propping or some irreversible initial deformation, because the size of unexplainable experimental error must be known to assess the experimental technique. That was not true of a structural design because what was then required was the lowest likely failure load. Thus, if there were two modes possible corresponding to different initial eccentricities, the lowest must be taken since there was a good probability of its occurrence. Moreover, there were often modes due to initial eccentricity which remained stable up to fairly high loads and then suddenly flicked over into the lowest. This could be tested with a simple pin-ended strut. If a fairly long strut (to prevent plastic intervention) was tested it would be found to fail repeatedly in one direction. If this strut were given an initial deflection in the opposite direction by a moment applied at one end, it would usually be found that the strut flicked over into the original critical direction before the critical load was reached.

The common mathematical foundation of apparently different methods in elastic analysis (including critical load analysis) was well known. Most methods of solution of a problem in fact could be reduced to the solution of a set of simultaneous equations no matter what philosophy was used. Thus the elastic analysis of a simple portal frame might be obtained for instance by strain energy, slope deflection, theorem of three moments, column analogy, moment distribution or relaxation. All were methods of solution of similar basic equations but some would be easier or quicker to use than others.

There was one important difference, however, between the convergence technique of this paper and most other methods of critical load analysis. All methods for other than trivial problems involved guessing a value of load parameter and checking that either a particular moment or rotation did not become negative or infinite or that some particular stiffness did not become zero in a specified calculation. With all other methods of which Dr. Bolton had had experience it was possible occasionally to obtain a higher value than the lowest critical load if the initial guess was more than a little too high, or the deformation was unluckily chosen. To take the simplest example, in the case of a pin-ended strut if the test was made at round about 2·05 or four times the Euler load, the calculation produced these as the solution instead of the true value, unity. These were answers on the dangerous side of the critical load, and although the answer was obviously wrong for a pin-ended strut, the corresponding answer might not be obviously wrong for a more complicated structure. The convergence technique, however, showed divergence continuously above the Euler load, and even if the initial guess were 3·99 times the Euler load the calculation would show the solution to be the Euler load. Thus the critical loads were not ' still given by $A = A_1, A_2 \ldots$'

This was because Mr. Gregory's set of equations (5) did not apply to the convergence method. The equalities given were true only for the (lowest) critical load A_1 since corrections to the value of $M_{o1}, M_{o2} \ldots$ then became zero in every cycle. Above the critical load there were no equalities since $a_1 M_1 + a_2 M_2 + \ldots$ became different from M_{o1} by a different amount in every cycle. The values of $M_{o1}, M_{o2} \ldots$ were then constantly modified cycle by cycle (with at least one of them tending to infinity) so (5) was not a set of equations which could be solved to represent the convergence method.

Dr. Bolton was glad to see Mr. Gregory's reference[7] to the paper by H. G. Allen. Dr. Allen carried out this work in the Department of Civil Engineering at Liverpool where Dr. Bolton was doing his own work, and Dr. Allen's method was the exact complement of the method presented in the present paper. The convergence method approached the critical mode by successive correction from initial rotations of all joints and stopped when enough cycles had been carried out to test convergence. Allen's method generated the critical mode exactly, starting with the rotation of one joint. His test was completed when the calculation arrived at the last joint.

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Institution Notices and Proceedings

HONORARY MEMBERSHIP

The Rt. Hon. Viscount Hailsham, P.C., Q.C., was elected an Honorary Member of the Institution on 28th April, 1960.

ORDINARY GENERAL MEETING

An Ordinary General Meeting of the Institution of Structural Engineers was held at 11, Upper Belgrave Street, London, S.W.1, on Thursday, 26th May, 1960, at 5.55 p.m., Mr. L. E. Kent, B.Sc.(Eng.), M.I.Struct.E., M.I.C.E. (President), in the Chair.

The following members were elected in accordance with the Bye-Laws. Will members kindly note that elections, as tabulated below, should be referred to when consulting the Year Book for evidence of membership.

Students

Bailey, Norman, of Keighley, Yorkshire.
Hainsworth, Winston Edward, of Transvaal, South Africa.
Henderson, Graeme Alan, of Ealing, Middlesex.
Hoult, Michael Robert, of Wellington, New Zealand.
Jefford, Nicholas John, of London.
Maguire, Robert, of Belfast, Northern Ireland.
Medak, George, of London.
Morris, Neil Alexander, of Wellington, New Zealand.
Perkins, John Shepherd, of Wellington, New Zealand.
Roberts, Frederick William, of Port Elizabeth, South Africa.
Tydeman, Alan, of London.

Graduates

Alderton, Arthur Francis, of London.
Avis, Peter Derek, of South Merstham, Surrey.
Beaman, Jack, M.Sc., of Heald Green, Cheshire.
Bell, Colin Walter Herbert, of Abbots Langley, Herts.
Bowers, Brian Richard, of Codnor, Derbyshire.
Choudhuri, Manju Kumar, B.Eng., of London.
Chown, Frank Edward, of Godalming, Surrey.
Clough, Norman, of Newton, Chester.
Colyer, Bryan Henry, of Beckenham, Kent.
Cruddas, Edward Alan, B.Sc., of Beckenham, Kent.
Das Gupta, Apurba Kumar, B.Eng., of Manchester.
Davis, Charles James, of Woking, Surrey.
Dixon, Paul Antony, of Orpington, Kent.
Drake, Michael John, of Kingswood, Bristol.
Farrar, Neil Shand, M.Sc., of Reigate, Surrey.
Gibbons, Barrington, of Thornton Heath, Surrey.
Haldar, Narayan Chandra, B.E., of West Bengal, India.
Harris, John Frank, of South Harrow, Middlesex.
Harris, Peter Leslie, B.Sc., of Gillingham, Kent.
Hone, Michael Vernon, of Southampton, Hants.
Kangatheran, Canagasundram, B.Sc., of London.
Kohli, Sudersham Kumar, B.Sc., of London.
Ledingham, Leslie William Clark, of Johannesburg, South Africa.
McDowell, Ronald George, of Tutshill, Nr. Chepstow, Monmouthshire.
McGrath, Frederick James, B.Sc., of Richmond, Surrey.
McQueenie, Anthony Francis, of North Harrow, Middlesex.
Makhijani, Mohan Atmaram, B.E., of London.
Mann, Anthony Frederick, of Bristol.
Mukhopadhyay, Ameresh, B.Eng., of London.
Nutt, Edward Daniel, B.Sc., of Wembley Park, Middlesex.
Omer, Hussein, of Liverpool.
Palmer, Gerald Florence, B.E., of London.
Parrott, Thomas Victor, of Liverpool.
Peake, Alan John, of Stafford.
Poh Kim Pew, of London.
Poole, James John, B.E., of London.
Povey, Anthony Thomas, of Mitcham, Surrey.
Prabhu, Sudhakar Shrirang, B.Eng., of London.
Ramachandran, K. N., B.Eng., of New Delhi, India.
Scrutton, Frederick Alma, D.A., A.R.I.B.A., of Edinburgh.
Sen, Satyabrata, B.Eng., of Calcutta, India.
Senior, John Robert, B.Sc., of London.
Si Hoe, Kok Keng, B.Eng., of Birmingham.
Stedman, Michael John, B.Eng., of Worcester Park, Surrey.
Tan Sin Eng, B.Sc., of London.
Thacker, Richard William, B.Sc., Tech., of Bramhall, Cheshire.
Thomas, Edwin Richard, B.E., of London.
Thomas, Richard Arthur Philip, of Bridgend, Glamorgan.
Thorp, Terence Norman, of Waltham Abbey, Essex.
Tonge, Geoffrey Harold, B.Sc., A.M.I.C.E., of Hazel Grove, Cheshire.
Wilford, Malcolm James Charles, of Sutton, Surrey.
Williams, John Hugh Hammerton, of London.
Woung, Clinton Rudyard, of Kew, Surrey.

Associate Members

Atkar, Dilbag Singh, B.Sc., of Belfast.
Limbada, Ismael Ahmad, B.E., of Vacoas, Mauritius.
Oraon, Kartik, B.Sc., of London.
Sinha, Thakur Munendra Nath, M.Sc., of London.
Spacey, Geoffrey, B.Sc.(Eng.), of Derby.

Member

Szeto, Wai, B.Sc., M.I.C.E., A.M.I.Mech.E., of Hong Kong.

Transfers

Students to Graduates

Jepson, Wilfred John, of Rusholme, Manchester.
Jones, David, of London.
McKelvie, John Gordon, of Stanmore, Middlesex.
Williams, David Powell Essex, of Cheadle, Cheshire.

Graduates to Associate-Members

Amodia, Rajnikant Mulchand, B.E., of London.
Buckton, Geoffrey, of Wetherby, Yorkshire.
Butler, Alan, of Manchester.
Butler, Robert, of Manchester.
Chan Kai Ming, of London.
Chanda, Sankar Lal, of London.
Ghosh, Utpal Krishna, B.E., of London.
Hewitt, Lawrence John, B.E., of London.
Jahina, Kaiomarz, B.E., of London.
Keys, Gerald, of Isleworth, Middlesex.
Lowe, John, B.Sc., of Chaddesden, Derby.
McArdle, Brian, of Salford, Lancashire.
Osborne, Ivan William, B.Sc., of London.
Pye, Ralph, of Thelwall, Nr. Warrington, Lancs.
Robson, Keith, of Greenford, Middlesex.
Searle, Michael George, of Langley, Slough, Bucks.
Sulley, William David, B.Eng., of Sunderland, Co. Durham.
Titchmarsh, Ronald John, of Liverpool.
Tonge, Derek George, of Stockport, Cheshire.
Topping, Jack, of Preston, Lancs.

Associate-Members to Members

BARKER, Walter Wilson, of Brighton, Sussex.
BARRY, Maurice Henry Alfred, of Elstree, Herts.
DE SOUZA, Ronald Cajetan, M.Sc.,(Eng.) B.Eng., A.M.I.C.E., of Twickenham, Middlesex.
ELDER, Frank, of Sale, Cheshire.
FINN, Edward Vivian, A.M.I.C.E., of London.
KING, John Henry Garrood, M.I.C.E., of London.
MCCLUSKY, Henry, A.M.I.C.E., of Glasgow.
MILLIKEN, Duncan, B.Sc.(Eng.), M.I.C.E., of Bakewell, Derbyshire.

Member to Retired Member

ORR, John Norman, M.B.E., of Herne Bay, Kent.

Associate to Retired Associate

HARRIS, Capt. Douglas, F.R.I.C.S., of London.

OBITUARY

The Council regret to announce the deaths of Lt.-Col. John Laurence FEAR (Associate), Arthur Benjamin DAILEY and Frederick Arthur GIBSON (Members), Watson GARBUTT and Robert ORMISTON (Associate Members).

RESIGNATION

Notification was given that the Council had accepted with regret the resignation of Frank Barford (Member).

ANNUAL GENERAL MEETING

The Annual General Meeting of the Institution of Structural Engineers was held at 11, Upper Belgrave Street, London, on Thursday, May 26th, 1960, at 6 p.m. Mr. L. E. Kent, B.Sc.(Eng.), M.I.Struct.E., M.I.C.E., in the Chair.

The Secretary (Major R. F. Maitland, O.B.E.), read the notice convening the meeting.

The Minutes of the Annual General Meeting held on May 28th, 1959, as published in THE STRUCTURAL ENGINEER, July, 1959, were taken as read and were confirmed and signed.

Lt.-Colonel G. W. Kirkland, M.B.E.(Mil.), (Vice-President), moved the adoption of the Sessional Report of the Council and the accounts for the financial year, 1959.

Mr. T. C. Waters (Delegate Member of Council), seconded the motion, which was carried unanimously.

Dr. D. D. Matthews (Vice-President), proposed the re-election of Messrs. James Meston & Co., Chartered Accountants, as Auditors for the ensuing year. Mr. A. L. Percy (Member of Council), seconded the motion, which was carried unanimously.

The Secretary then read the report of the Scrutineers on the ballot for the election of President, the Honorary Officers and the Ordinary members of Council for the Session 1960-61, as follows :— To : The Council of the Institution of Structural Engineers.

Gentlemen :

We, the undersigned, report that at the request of the President we have duly carried out the duties of Scrutineers of the Ballot for the election of Honorary Officers and Council for the Session 1960-61, and we report accordingly as follows :—

We received 827 Ballot Papers, of which we rejected two as wholly spoiled and 12 as partly spoiled. We have attached a separate sheet showing the number of votes received by each candidate.

We declare the result of the Ballot to be as follows :—

ELECTED

President : Lt.-Col. G. W. Kirkland, M.B.E.(Mil.), M.I.C.E.

Vice-Presidents: Mr. F. R. Bullen, B.Sc.(Eng.), M.I.C.E., Mr. E. N. Underwood, B.Sc.(Eng.), M.I.C.E., Mr. F. M. Bowen, M.I.C.E., Assoc.I.Mech.E., Mr. H. C. Husband, B.Eng., M.I.C.E., M.I.Mech.E., Dr. D. D. Matthews, M.A., M.Sc.(Eng.), M.I.C.E., Mr. John Mason, B.A.(Cantab), A.M.I.C.E.

Honorary Treasurer : Mr. D. A. G. Reid, B.Sc.(Eng.), M.I.C.E.

Honorary Secretary : Mr. B. Scruby

Honorary Librarian : Dr. F. G. Thomas, B.Sc.(Eng.), M.I.C.E.

Honorary Editor : Mr. G. C. A. Greetham, O.B.E.

Honorary Curator : Mr. Donovan H. Lee, M.I.C.E., M.I.Mech.E.

The above are all elected for ONE YEAR.

Elected as Ordinary Members of Council (*London*) : Mr. C. B. Stone, D.S.O., B.Sc., M.I.C.E., Mr. Morris T. Shaw, B.Sc., (Eng.) M.I.C.E., Mr. D. R. R. Dick, B.Sc., M.I.C.E.

The above are elected for THREE YEARS.

Elected as Ordinary Member of Council (*Country*) : Mr. T. C. Waters.

The above is elected for THREE YEARS.

Elected as Associate-Member of Council (*London*) : Mr. J. A. Derrington, B.Sc., (Eng.) M.I.C.E.

The above is elected for THREE YEARS.

Elected as Associate-Member of Council (*Country*) : Mr. D. W. Cooper, B.Sc.(Hons.).

The above is elected for THREE YEARS.

We are, Gentlemen,
Yours faithfully,
(*Signed*) R. D. McMeekin
F. E. Fowle
E. O. Measor
W. Pring.

On a motion by the President, a vote of thanks was unanimously passed to the scrutineers.

EXAMINATIONS, JULY, 1960

The Examinations of the Institution will next be held in the United Kingdom and overseas on Tuesday and Wednesday, July 12th and 13th, 1960 (Graduateship) and Thursday and Friday, July 14th and 15th, 1960 (Associate-Membership).

DRURY MEDAL AWARD—1961

The alternative subjects for this the eighth Competition, are a pipe-bridge and a petrol service station.

Graduates and Students of the Institution not over 27 years of age are invited to apply for full details to the Secretary, envelope to be marked in the top left-hand corner ' Drury Medal Award.'

The closing date for the competition is October 1st, 1961.

The general conditions of the competition are as follows :

(a) The competition shall be for Graduates and Students of not more than 27 years of age.
(b) The subjects of the competition will be designs of a structural character, that is to say, involving structural design rather than planning.
(c) The subjects of design and the conditions shall be prepared and issued biennially.
(d) A Jury shall be appointed to examine the work submitted and to interview candidates if found necessary.
(e) In order to ensure that the design submitted is the unaided work of the competitor, the drawings, calculations, etc., submitted shall be endorsed by the candidate : " I declare that the work I hereby submit is my own unaided work." The declaration shall be signed by the competitor,

and be either countersigned by a corporate member, or be certified as made before a Justice of the Peace, or a Commissioner for Oaths.

Branch Notices

LANCASHIRE AND CHESHIRE BRANCH

The following Honorary Officers and Committee members have been elected for the Session 1960-61 :—
Chairman : Professor W. Merchant.
Vice-Chairman : J. E. Guest.
Immediate Past Chairman : T. C. Waters.
Honorary Secretaries : W. S. Watts, 11, Newchurch Lane, Culcheth, Nr. Warrington, Lancs. and M. D. Woods. 8. Dennison Road, Cheadle Hulme, Cheshire.
Committee Members : W. Bates, Professor A. W. Hendry, K. Norrey, F. C. Brookhouse, J. S. Watts, H. Heaton, R. Gray, S. Niele, J. B. Story, A. G. McNamara, A. S. Sinclair, W. Fitton.
Honorary Auditors : K. Norrey and G. S. Jones.
Merseyside Panel Committee : Honorary Secretary, B. N. Wright ; Committee, Professor A. W. Hendry, A. G. McNamara, J. W. Baron, E. Thorpe.

MIDLAND COUNTIES BRANCH

Hon. Secretary : S. M. Cooper, A.M.I.Struct.E., "Applegarth," Hyperion Road, Stourton, Nr. Stourbridge, Worcestershire.

GRADUATES' AND STUDENTS' SECTION

Hon. Secretary : H. T. Dodd, Shepherd's Cottage, Grove Lane, Wishaw, Sutton Coldfield, Warwickshire.

NORTHERN COUNTIES BRANCH

Hon. Secretary : P. D. Newton, B.Sc., A.M.I.Struct.E., A.M.I.C.E., c/o Cusson & Partners, 113, Borough Road. Middlesbrough, Yorkshire.

NORTHERN IRELAND BRANCH

The Annual General Meeting of the Branch was held on 5th April, when the following Honorary Officers and Committee members were elected for the Session 1960-61.
Branch Chairman : W. A. Plester.
Vice Chairman : J. H. Currie.
Immediate Past Chairman : A. H. K. Roberts.
Honorary Auditors : J. Chrisp, R. E. Moore.
Honorary Secretary : L. Clements, 3, Kingswood Park, Cherryvalley, Belfast, 5, Northern Ireland.
Honorary Assistant Secretary : J. D. McCaughey.
Committee : M. Agar, J. D. Boyd, S. O. Morton, P. S. Rhodes, S. Taggart.

SCOTTISH BRANCH

Hon. Secretary : W. Shearer Smith, M.I.Struct.E., A.M.I.C.E., c/o The Royal College of Science and Technology, George Street, Glasgow, C.1.

SOUTH WESTERN COUNTIES SECTION

Hon. Secretary : C. J. Woodrow, J.P., "Elstow," Hartley Park Villas, Mannamead, Plymouth, Devon.

WALES AND MONMOUTHSHIRE BRANCH

The following Honorary Officers and Committee members have been elected for the Session 1960-61 :—
Chairman : W. H. Bartlett.
Vice Chairmen : E. Hughes, K. J. Stewart.
Honorary Secretary : W. D. Hollyman, 41 Greensfield Avenue, Dinas Powis, Glamorgan.
Honorary Assistant Secretaries : South Wales, G. W. Spooner ; North Wales, A. Hammerton.
Honorary Auditors : G. W. Spooner, E. R. Steward.
Committee : J. L. Bannister, J. M. Burke, G. R. Brueton, G. E. Cooper, W. A. Evans, G. H. Hodgson, A. V. R. Hooker, D. A. Lewis, D. Manolopoulos, G. W. Spooner, E. R. Steward, A. G. Thompson, G. Williamson.
Ex Officio Members : W. H. Bartlett, E. Hughes, K. J. Stewart, W. D. Hollyman, A. Hammerton.

WESTERN COUNTIES BRANCH

The following Honorary Officers and Committee members have been elected for the Session 1960-61 :—
Chairman : Clifford E. Saunders.
Vice-Chairman : F. G. Clarke.
Hon. Vice-Chairman : C. H. Williams.
Hon. Secretary : A. C. Hughes, 21 Great Brockeridge, Bristol, 9.
Hon. Treasurer : F. K. Fennell.
Hon. Assistant Secretary : D. J. Bickerton.
Immediate Past Chairman : Peter Gardiner, *Ex officio* member of Committee in accordance with Bye-Law 22.
Honorary Auditors : F. A. Long, J. M. Rome.
Committee : N. G. T. Ball, R. H. Barnett, D. Bond, J. C. Colling, C. J. D. Boxall, A. J. Brownston, R. G. H. Jenkins, L. A. Macer, G. C. Mander, E. N. Underwood, A. W. Wall, S. T. Wyatt.

YORKSHIRE BRANCH

The following Honorary Officers and Committee members have been elected for the Session 1960-61 :—
Chairman : Dr. W. Eastwood.
Senior Vice-Chairman : T. E. S. White.
Junior Vice-Chairman : I. H. Paxton.
Immediate Past Chairman : Dr. E. Lightfoot.
Hon. Auditors : C. C. Begg, F. A. Maddock.
Hon. Secretary : W. B. Stock, 34 Hobart Road, Dewsbury, Yorks.
Committee : A. C. L. Blake, T. F. Cliffe, John Dossor, H. W. Dowe, H. C. English, T. Holmshaw, J. Jenkins, F. A. Maddock, E. J. Martin, A. L. Percy, L. Preston, J. P. Robinson, L. C. Walker, Capt. F. A. Whitaker, D. R. S. Wilson.
Ex Officio : Professor J. Husband (Past President).

UNION OF SOUTH AFRICA BRANCH

Hon. Secretary : A. E. Tait, B.Sc., A.M.I.Struct.E., A.M.I.C.E., P.O. Box 3306, Johannesburg, South Africa.

During weekdays, Mr. Tait can be contacted in the City Engineer's Department, Town Hall, Johannesburg. Phone 34-1111, Ext. 257.
Natal Section Hon. Secretary : J. C. Panton, A.M.I.Struct.E., A.M.I.C.E., c/o Dorman Long (Africa) Ltd., P. O. Box 932, Durban.
Cape Section Hon. Secretary : R. F. Norris, A.M.I.Struct.E., African Guarantee Building, 8, St. George's Street, Cape Town.

EAST AFRICAN SECTION

Chairman : R. A. Sutcliffe, M.I.Struct.E., P.O. Box 30079, Nairobi, Kenya
Hon. Secretary : K. C. Davey, A.M.I.Struct.E., P.O. Box 30079, Nairobi, Kenya.

NIGERIAN SECTION

Chairman : J. W. Henderson, E.R.D., B.Sc., M.I.Struct.E., M.I.C.E.
Hon Secretary : A. Brimer, A.M.I.Struct.E., Brimer, Andrews and Nachshen, Private Bag Mail 2295, Lagos, Nigeria.

SINGAPORE AND FEDERATION OF MALAYA SECTION

Chairman : T. F. Lee, B.Sc.
Hon Secretary : W. N. Cursiter, B.Sc., A.M.I.Struct.E., A.M.I.C.E., c/o Redpath Brown and Co. Ltd., P.O. Box 648, Singapore.

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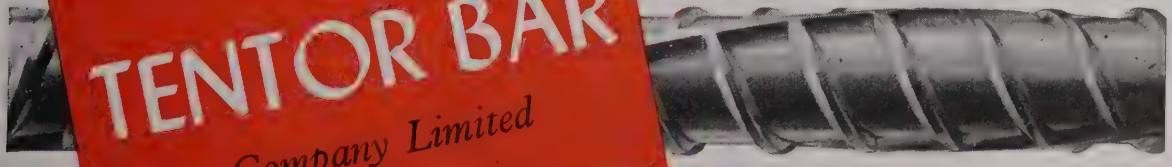
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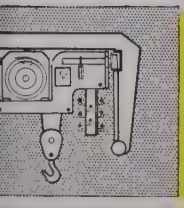
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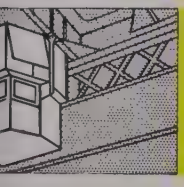
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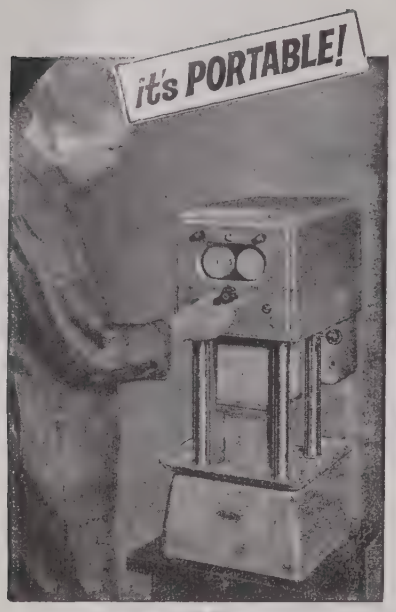
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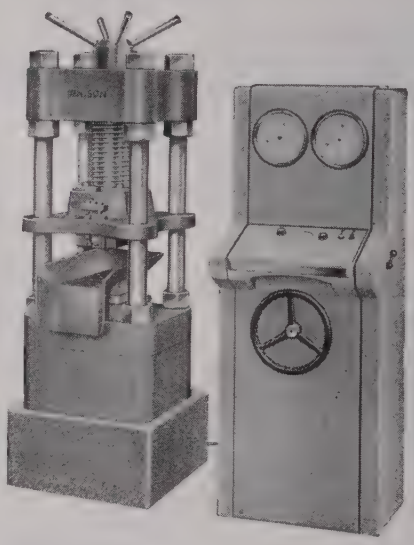
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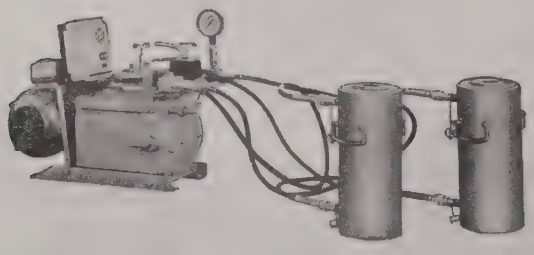
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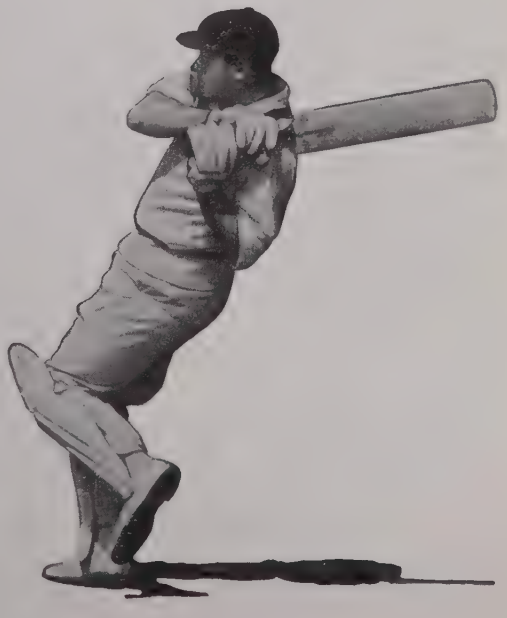
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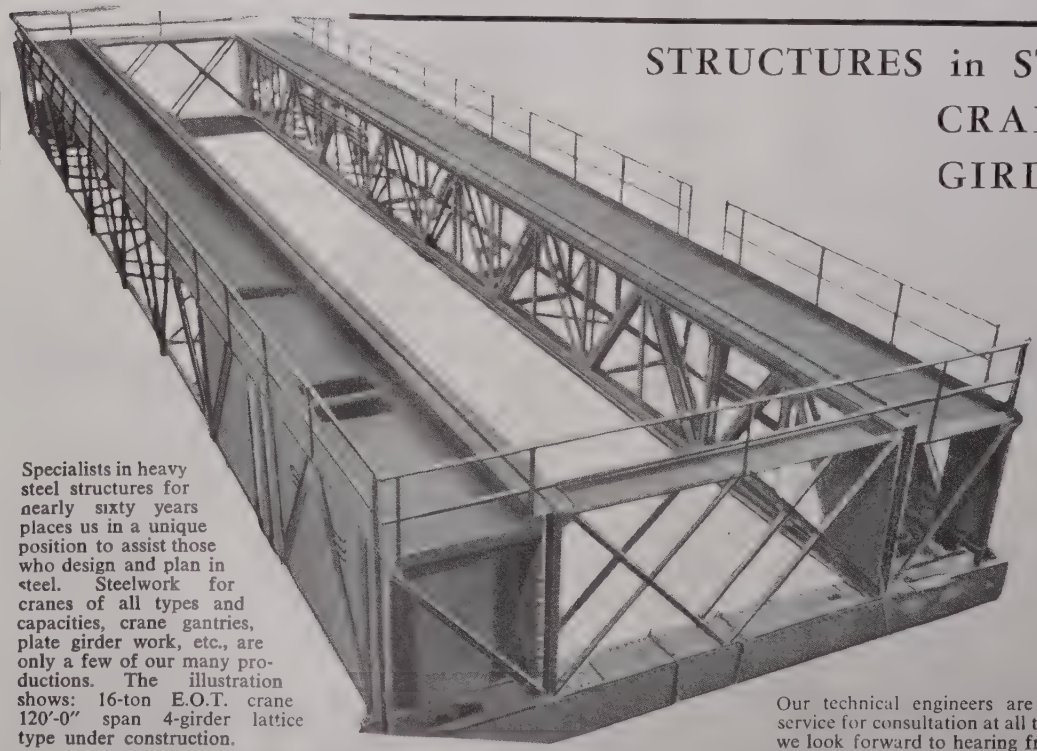
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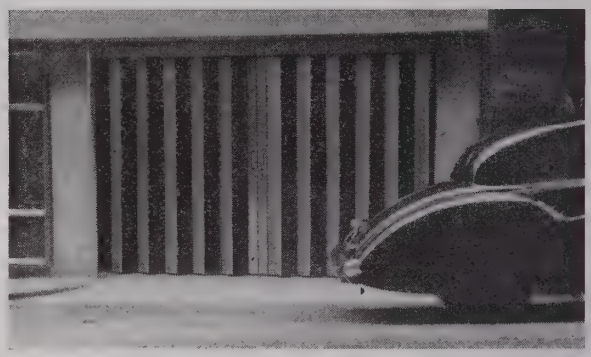
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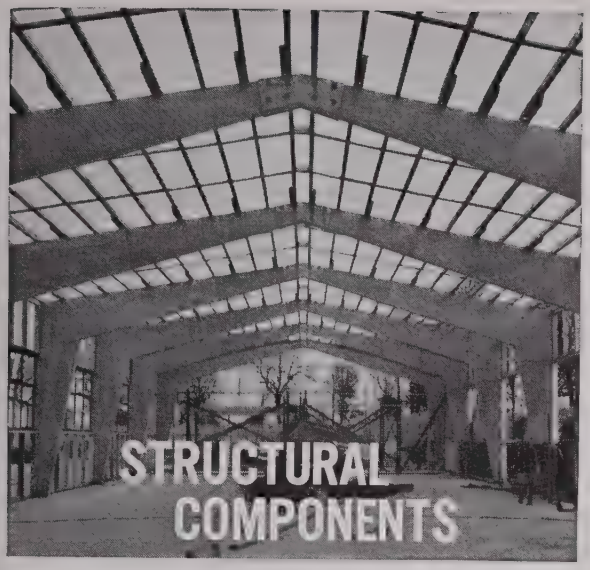
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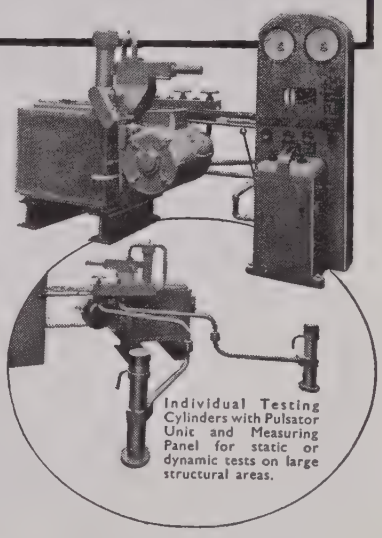
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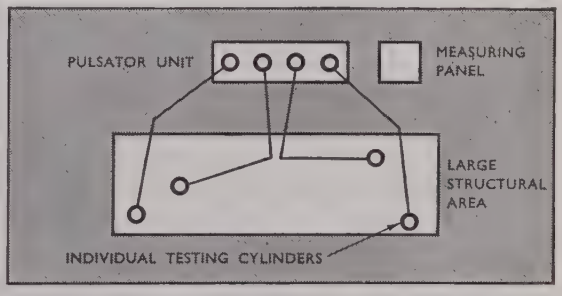
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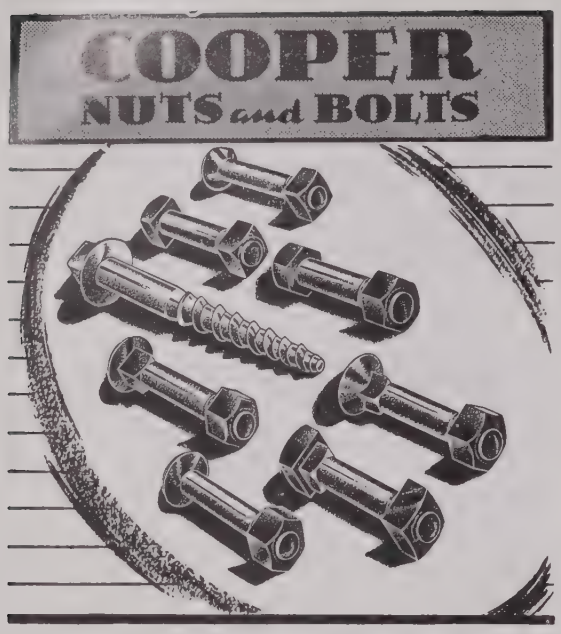
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INDEX TO ADVERTISERS

	Page
Associated Lead	18
Arrol, Sir William & Co. Ltd.	17
Brookes & Co. (1925) Ltd.	42
Braithwaite & Co. (Engineers) Ltd.	9
British Reinforced Concrete Engineering Co. Ltd., The	Cover IV
British Oxygen Gases Ltd.	4 & 25
Booth, John & Sons (Bolton) Ltd.	13
British Steel Piling Co. Ltd.	26
Bell & Webster Ltd.	42
British Industrial Engineering Company (Staffs) Ltd.	35
Beves & Co. (Structures) Ltd.	24
Cooper, George & Sons	46
Crompton Parkinson (Stud Welding Ltd.)	8
Contract Journal, The	2
Cement Marketing Co. Ltd. The	22
Consett Iron Co. Ltd.	38
Costain Concrete Co. Ltd.	40
Denison, Samuel & Son Ltd.	36
Dawnays Ltd.	16
Equipment & Engineering Co., The	36
Frankipile Ltd.	19
Ground Explorations Ltd.	10
Hill, Richard Ltd.	Cover II
Holoplast Ltd.	5
Johnson, Richard & Nephew Ltd	12
Kennedy, Allan & Co. Ltd	Cover III
Lind, Peter & Co. Ltd.	28
Laing, John & Son Ltd.	32
Lytag Ltd.	21
Lafarge Aluminous Cement Co. Ltd.	29
McCall & Co. (Sheffield) Ltd.	20
McKinney Foundations Ltd.	30
Mellowes & Co. Ltd.	14
Plywood Association of Great Britain	43
Standard Telephones & Cables Ltd.	23
Stramit Boards Ltd.	41
Skipwith, Jones & Lomax	38
Shell International	27
South Durham Steel & Iron Co. Ltd.	7
Simon-Carves Ltd.	3
Smith, John (Keighley) Ltd.	6
Simplex Concrete Piles Ltd.	10
Skinningrove Iron Co. Ltd.	11
Site Precast Concrete Ltd.	37
Tentor Bar Co. Ltd. The	33
Tileman & Co. Ltd.	39
Thermalite Ytong Ltd.	15
Town, Fredk. & Sons Ltd.	34
Ward, Thos. W. Ltd.	1
Wickman Ltd.	43

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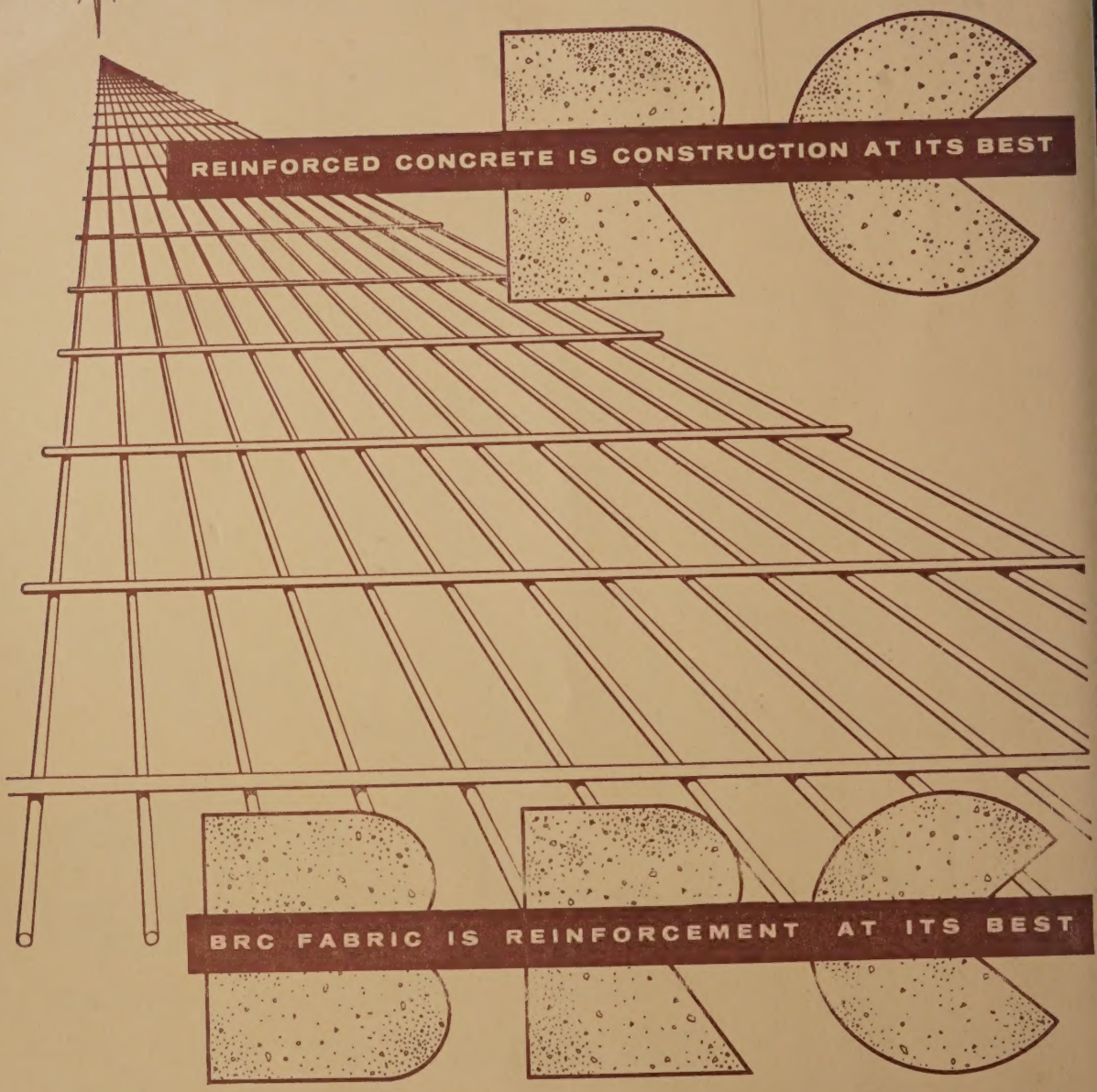
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